JOURNAL OF THE INSTITUTION OF CIVIL ENGINEERS.

No. 3. 1936–37.
JANUARY 1937.

ORDINARY MEETING.

1 December, 1936.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

Sir NOEL ASHBRIDGE, B.Sc. (Eng.) (*Lond.*).
JAMES ARTHUR BANKS.
JOHN ERIC STURTON BODGER
CYRIL WALTER CARTER.
JAMES CONACHER.
CLEMENT HENRY CRUTTWELL.
WILLIS DIXON.
ERIC CARLEN FAWCETT.
REGINALD GADSBY.
HAROLD MARRIOT GELL, M.C.
THOMAS CHARLES SKEFFINGTON HASLAM.
GEORGE HESELDIN, M.Sc. (*Leeds*).
WILLIAM HUDSON, B.Sc. (Eng.) (*Lond.*).
ALBERT WILLIAM JACKSON.
JULIUS KENNARD, B.Sc. (Eng.) (*Lond.*).
GERALD LACEY, B.Sc. (Eng.) (*Lond.*).
WILLIAM ALLAN MACARTNEY.
DANIEL GEORGE MACCORMACK, B.Sc. (*Glas.*).
MAURICE NACHSHEN, B.Sc. (Eng.) (*Lond.*).
WILLIAM JAMES PRICE.
HUBERT SHIRLEY SMITH, B.Sc. (Eng.) (*Lond.*).
JOHN BELL LANGHORN THOMPSON, M.C.
JOHN THOMSON, B.Sc. (*Glas.*).
Sir HAROLD EDGAR YARROW, Bart., C.B.E.

And had admitted as

Students.

HAROLD JOHN ABSALOM, B.A. (*Cantab.*).
PHILIP ARCHIBALD ALLISON, B.Eng. (*Sheffield*).
JOHN EDWARD HENRY ANDERSON.
JAMES MACQUARIE ANTILL, B.E. (*Sydney*).
GEORGE STANLEY BARKER, B.Sc. (*Birmingham*).
ERIC EUGENE BRUCE BARTON.
JAMES KELL LAMIE BLACK, B.Sc. (*Belfast*).
PETER HENRY BODGENER.
ALAN BRIERLEY, B.Sc. Tech. (*Manchester*).
DERYCK BROWN.
FRANK BERTRAM BULL.
HENRY ANTHONY VAUGHAN BULLEID, B.A. (*Cantab.*).
THOMAS WILLIAM CALVERT.
JOHN CHRISTIE.
NORMAN WALTER WALKER COCKCROFT, B.Eng. (*Liverpool*), M.Sc. (*Cantab.*).
JOHN STUART COLMAN.
MERVYN MORGAN DAVIES, B.Sc. (*Wales*).
KENNETH DENING DUDLEY, B.Sc. (Eng.) (*Lond.*).
BIBHUTI BHUSHAN DUTTA, B.C.E. (*Patna*).
GAURCHANDRA GHOSH, B.E. (*Calcutta*).

John Darlow Gleeson.
Hugh Sidney MacKay Graham, B.Sc. (*Glas.*).
Thomas Dugald Gray.
Basil Lyndhurst Colton Haines, B.Sc. (*Durham*).
John Jackson Hamilton, B.A. (*Cantab.*).
Jack Malcolm Harness, B.Sc. (*Manchester*).
John Hayes, B.Sc. (*Manchester*).
Paul Woodrow Hogben.
James Edgar Holme.
John Scott Hutton.
Alexander Russell Jardine, B.A. (*Cantab.*).
Royston Jones, M.Sc. Tech. (*Manchester*).
Morris Philip Kossansky.
Robert Leslie, B.Sc. (*Cape Town*).
William Allan Probert McColville.
Sinclair James McKay.
James William MacLeod, B.Sc. (*Aberdeen*).
Francis Sumner Matthews, B.Sc. (*Bristol*).
Jean Laurence Nairac, B.A. (*Cantab.*).
Kevet Michael Hastings Neale.
Robert Keith Nicholson, B.A. (*Cantab.*).
Cyril Oldfield, B.Sc. Tech. (*Manchester*).
Paul Charles Peppiette, B.Sc. (*Birmingham*).
Edward Patrick Purcell, B.A.I. (*Dubl.*).
Thomas Herbert Robinson, B.Sc. Tech. (*Manchester*).
Ralph Leslie Rolph, B.A. (*Cantab.*).
William Rutherford, B.Sc. (*Durham*).
Charles Robert Sanders.
Norman Howorth Schofield.
Mysore Seshadriengar, B.E. (*Mysore*).
Francis Eric Sheals.
William Smart, B.Sc. (*Edin.*).
John Smith, B.Sc. (*Edin.*).
James Arthur Stephenson, B.Sc. (*Durham*).
John Stewardson.
Alastair Storrar, B.Sc. (*Edin.*).
Oliver Midgley Taylor.
Lindsey Eric Thompson.
Peter Hugh Frederick Walker.
Brian Cyril Warner, B.A.(*Cantab.*).
Ian Edward Main Watts, M.Sc. (*New Zealand*).
Wilfrid Welch, B.Sc. (*Durham*).
Enoch Wharmby, B.Sc. Tech. (*Manchester*).
Aubrey White.
Alan Wicksteed.
Andrew Wilson, B.Sc. (*Aberdeen*).
Edwin Douglas Winton.
Stanley Whyte Wood, B.Sc. (*Aberdeen*).

The Scrutineers reported that the following had been duly elected as

Members.

Professor Charles Andreae.
Arthur Herbert Andrews.
John Ashurst.
Herbert Joseph Crofts.
Joseph Darius Griffin.
Benjamin Handley.
John Allan Thornton.
William Downing Woolley.

Associate Members.

Mazhuvanchery Chakko Abraham, B.Sc. (*Manchester*).
Sheikh Basheer Ahmed, M.Sc. (*Hong Kong*).
Robert Fyfe Alexander.
Ronald Charles Tucker Allen, M.A. (*Cantab.*), Stud. Inst. C.E.
Alan Geoffrey Allnut, B.Sc. (Eng.) (*Lond.*).
James William de Alwis.
James Hamilton Anderson, B.Sc. (*Glas.*).
Howard Archer, Stud. Inst. C.E.
Donald Dunmore Ash, Stud. Inst. C.E.
Arnold Atherton, B.Sc. (*Manchester*).
Brian Bernard Atherton, M.A. (*Cantab.*), Stud. Inst. C.E.
William Atkinson.

Frederick Cornelius John Bagnall, B.Sc. (*Birmingham*).
Frank Ralph Baines, M.A. (*Cantab.*), Stud. Inst. C.E.
William Thomas Baker, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
George William Thorold Barnett, B.Sc. (*Glas.*).
Arthur Frank Norman Barnsdale, Stud. Inst. C.E.
John Reynolds Barron.
Henry Hargreaves Bateson.
William James Berry.
Dennis Percy Bertlin, M.Eng. (*Liverpool*), Stud. Inst. C.E.
Alfred Maurice Binnie, M.A. (*Cantab.*), M.A. (*Oxon.*).
Thomas Blench, B.Sc. (*Glas.*).
Thomas James Boddington, B.Sc. (Eng.) (*Lond.*).
James Brearley, B.Sc. Tech. (*Manchester*), Stud. Inst. C.E.
Charles Bell Brown, B.A.I. (*Dubl.*).
John Logan Brown, B.Sc. (*Glas.*).
Nabih Yusif Bulos, Stud. Inst. C.E.
Arthur Henry Clarke, B.Sc. (*Wales*).
Frederick Earland Philip Clear, B.A. (*Cantab.*), B.Eng. (*McGill*), Stud. Inst. C.E.
Victor Alfred Collins, Ph.D., M.Sc. (*Birmingham*).
Robert Sutherland Colquhoun.
George Henry Harris Cook.
Richard Thomas Cooke.
Albert Sidney Coombs.
Harold William Cother, B.Sc. (Eng.) (*Lond.*).
Thaddeus Cornelius Courtney, M.E. (*National*).
Kenneth Henry Cowin, B.Sc. (*Manchester*), Stud. Inst. C.E.
Wilfred Frank Cox, B.Sc. (Eng.) (*Lond.*).
Henry Crichton, Stud. Inst. C.E.
James Smith Crichton, B.Sc. (*Belfast*), Stud. Inst. C.E.
Russell Mackenzie Currie.
George Dalkins, Stud. Inst. C.E.
David Bruce Dallas.
David Jones Davies, B.Sc. (Eng.) (*Lond.*).
Frank Dawson.
Victor Cyril De Silva, B.Sc. (Eng.) (*Lond.*).
John Samuel Duncan, B.Sc. (*Birmingham*).
John Percival Emerson.
Frank George Etches, B.Sc. (Eng.), (*Lond.*).
George Richard Evans.
Thomas Sinclair Fitzgerald, B.A., B.A.I. (*Dubl.*).
Robert Arthur Foulkes, M.A. (*Cantab.*), Stud. Inst. C.E.
John Harold William Freeman, B.Sc. (*Leeds*), Stud. Inst. C.E.
Basil Erle Gardner, B.A., B.A.I. (*Dubl.*).
Harry Norwood Ginns, Stud. Inst. C.E.
John Cubbon Grindley.
Robert John Hamilton, B.Sc. (Eng.) (*Lond.*).
Clifford Hardman.
John Harris, B.Sc. (Eng.) (*Lond.*).
Vernon Dunbavin Harty, B.A., B.A.I. (*Dubl.*).
Claude William Henderson.
Darrell Bertram Herbert.
Henry Alfred Christian Page Hetherington, Stud. Inst. C.E.
James Hodson, B.Sc. (Eng.) (*Lond.*).
Robert Hoggan, B.Sc. (Eng.) (*Lond.*).
Edmund Howard.
Michael George Ionides, B.A. (*Oxon.*).
James Jamieson, Jun.
Ralph Dennis Jennings, B.Sc. (*Bristol*).
Cecil Owen Innes Jones, B.Sc. (*Leeds*), Stud. Inst. C.E.
William Milfranydd Jones, B.Sc. (*Bristol*), Stud. Inst. C.E.
Francisco Joaquim Maria Jorge, B.Sc. (*Hong Kong*).
Don Wilfred Richard Kahawita, B.A. (*Cantab.*).
Mervyn Dixon Kharegat, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Jeffrey William Hitchen King, B.Sc. (*Manchester*).
Alan Forsyth Lambert, B.A. (*Cantab.*).
Peter Anton Lamont, M.A. (*Cantab.*), Stud. Inst. C.E.

Reginald Stone Ashley Larmuth, M.A. (*Cantab.*).
Edmund Arthur Leach, B.Sc. (*Leeds*), Stud. Inst. C.E.
George Vernon Lewis, B.Sc. (*Cape Town*), Stud. Inst. C.E.
Thomas Lightbown, Stud. Inst. C.E.
James Lindsay, Stud. Inst. C.E.
Robert Lindsay, B.Sc. (*Glas.*).
John Ford Sewell MacDonald.
George Holmes McKenzie.
Alfred Philip Mainprice, B.Sc. (Eng.) (*Lond.*).
Arthur Norrison Marr, Jun., B.Sc. (*Leeds*), Stud. Inst. C.E.
Basil Merry, B.Sc. (Eng.) (*Lond.*).
Thomas William Stanley Milligan.
William Morley-Scott, B.Sc. (Eng.) (*Lond.*).
James Robinson Musgrave, Stud. Inst. C.E.
Patrick Ibar O'Connor, B.E. (*National*).
Caradog Morris Owen, Stud. Inst. C.E.
Jack Palmer.
Barney Pattenden, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Frank William Pearce, Stud. Inst. C.E.
Leslie Martin Perkis.
Edmund Hugh Leigh Pierce, B.Sc. (*Wales*).
Ronald George Plummer, B.Sc. (*Durham*).
Ivor Powell.
Albert Mathieson Quennell, B.Sc. (*New Zealand*).
Robert Leslie Rankin.
Karunamay Ray, B.Sc. (*Calcutta*), B.Sc. (Eng.) (*Lond.*).
Samuel Melya Reisser, B.Sc. (Eng.) (*Lond.*).
Harold Charles Rowan, B.Sc., Ph.D. (*Birmingham*).
Ignatius John Harries Sabido, B.Sc. (*Wales*).
Frederick John Sanders, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
John Gordon Sansom, B.Sc. (*Glas.*).
Haydn Schofield, B.Sc. (Eng.) (*Lond.*).
Reginald Mac Whiteley Shaw, Stud. Inst. C.E.
Kalyanbhai Lalbhai Sheth, B.Sc. (Eng.) (*Lond.*), B.E. (*Bombay*).
William Hudson Shoesmith, Stud. Inst. C.E.
George Smethurst, B.Sc. (*Manchester*), Stud. Inst. C.E.
John Victor Spalding, B.A. (*Cantab.*), Stud. Inst. C.E.
Francis Henry Stokes, B.Sc. (*Bristol*), Stud. Inst. C.E.
Charles Thorp.
Robert Richard Tisdale.
Henry Mercer Tomlinson, Stud. Inst. C.E.
Sydney William Trueman, Stud. Inst. C.E.
Donald William Tull.
George Verity.
Arthur William Crips Villiers, B.A., B.A.I. (*Dubl.*).
James Davidson Wallace, B.Eng. (*Sheffield*).
Maurice Frederick Telford Ward, Stud. Inst. C.E.
Eric George Warrington.
John Simpson Watt.
Henry Waymouth, B.E. (*New Zealand*).
Algernon Wear, B.Sc. (*Leeds*).
Arthur Stanley West.
Maurice Charles West, B.Sc. (Eng.) (*Lond.*).
Hector William Westwood.
Donald Eacott Whiting, B.Sc. (Eng.) (*Lond.*).
Vernon Charles Whiting.
Edwin Peter Littlefear Wilders, B.Sc. (Eng.) (*Lond.*).
John Samuel Williams, Stud. Inst. C.E.
Paul Norman Wilson, M.A. (*Cantab.*).
Robert Wilson, Stud. Inst. C.E.
Eric Wongtape, B.Sc. (*Hong Kong*), Stud. Inst. C.E.
William James Wonnacott.
Herbert Wyles.

Associate.

Reuben Swager.

The following Papers were submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 5054.

"The Lower Zambezi Bridge." †

By Frederick William Adolph Handman, C.B.E., M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

The Lower Zambezi bridge is in Portuguese East Africa. Its western end is on the right bank of the river Zambezi at Sena in the Moçambique Company's territory, 198 miles north of the port of Beira, and its eastern end is at Dona Anna near Mutarara in the Colony of Moçambique, 152½ miles south of Blantyre in Nyasaland (*Fig. 1*, p. 327). The bridge was built under a concession from the Portuguese Government to the British Central Africa Company, the concession being subsequently transferred to the Central Africa Railway Company. In accordance with the terms of the concession the bridge will revert to the Portuguese Government in 99 years

† Correspondence on this Paper can be accepted until the 15th May, 1937. —Sec. Inst. C.E.

from the year 1912. The construction of the bridge and its connecting railways was financed from the Guaranteed Loan Fund authorized by the British Government in 1926 to assist in the economic development of the East African Colonies.

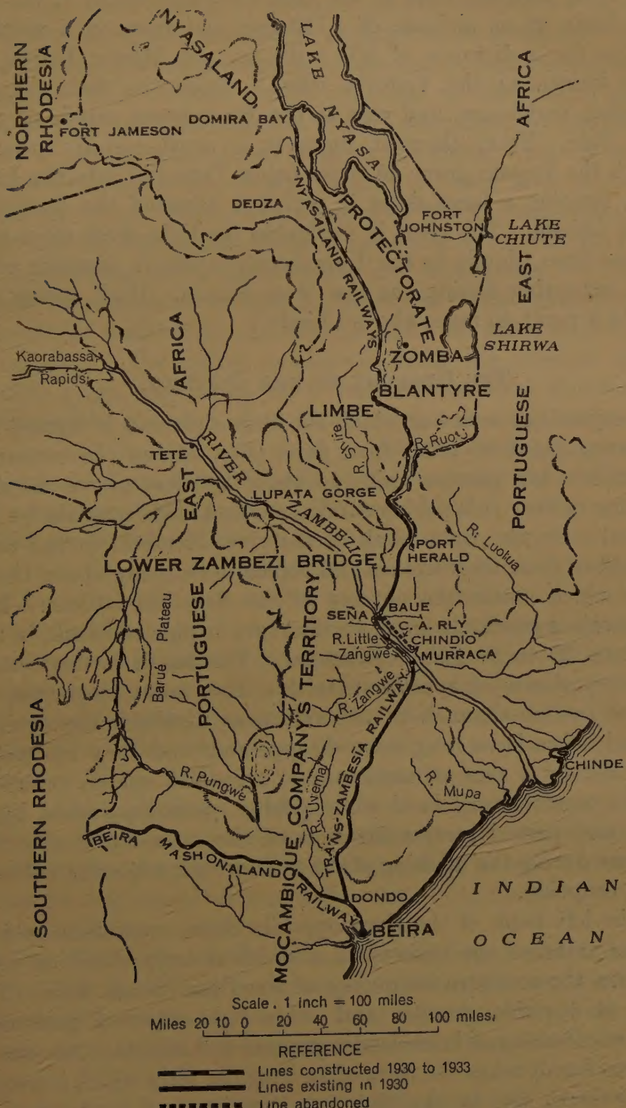
The Object of the Bridge.

The railways which formerly served Nyasaland ran from Beira to Murraça, and from Chindio to Blantyre, with a ferry service across the Zambezi from Murraça to Chindio (*Fig. 1*).

At certain points near Murraça on the right bank, and for 20 miles from Chindio on the left bank, the track was subject to inundation when the Zambezi was in high flood, with consequent damage to the permanent way and disorganization of traffic, whilst the ferry traffic across the river from Murraça to Chindio was carried on under very severe conditions and with great difficulty. The vagaries of the river Zambezi during the low-river season from the beginning of October to the middle of December rendered it impossible to establish wharves and permanent cranes on either bank for handling and transferring goods from rail to ferry, and merchandise had to be handled by hand with the assistance of small-capacity portable conveyors for sacked and baled goods. Owing to the rapid silting up of navigation-channels at low river it was necessary during some seasons to move the loading stations and traffic sidings to points as far apart as 3 miles. Sandbanks form in a night, and it frequently happened that ferry steamers had to be assisted across shallow places by big gangs of natives pushing and poling them.

The Guaranteed Loan Committee, after having investigated alternative schemes, therefore came to the conclusion that the construction of a bridge was the only practical method of solving the problem of the Zambezi crossing. The construction of the bridge involved the construction of two connecting railways, one on the south 25·29 miles long to connect the terminus of the Trans-Zambesia Railway at Murraça with the western end of the bridge, and the other on the north 2·91 miles long to connect the eastern end of the bridge with the Central Africa Railway at Baue. It also involved the abandonment of 24½ miles of the Central Africa Railway between Chindio and Baue, the permanent way and bridge material from which was used elsewhere. The object of the bridge was therefore to permit of a through system of railways from Beira to lake Nyasa, without which the larger development scheme of the Nyasaland Protectorate could not be carried out to advantage. The railway system in operation to-day is indicated in *Fig. 1*, its length being 527 miles.

Fig. 1.



GENERAL MAP.

The River Zambezi.

The river Zambezi rises in the Dilolo swamps south of the river Congo basin at an altitude of 4,750 feet and flows in a southerly direction for a distance of 800 miles to the Mosioa-Tunya falls (better known as the Victoria falls). From the falls the river flows east, north-east, and east to the Kaorabassa rapids 63 miles above Tete. From there it flows in a south-easterly direction through the Lupata gorge 45 miles below Tete and into the Indian Ocean, forming a large delta in which is situated the Portuguese port of Chinde. The river is navigable for light-draft stern-wheel steamers from Chinde to the Kaorabassa rapids (a distance of 340 miles), excepting during the low river season. High spring tides reach to a point 40 miles above Chinde.[1]

The Geology of the District.

The crystalline series of which the Barue plateau is formed is at the nearest point 60 miles from Sena (*Fig. 1*) and the country in between is a low plateau rising to about 200 feet above the river, consisting of very soft pebbly sandstones and clays overlain by beds of gravel. On the right bank of the river Zambezi in the neighbourhood of Sena there are several conical hills rising above the low ground which marks the outcrop of the Sena sandstones. These hills contain a relatively small vertical core or pipe of basalt, in most cases from 75 to 90 feet in diameter. The basalt is described as olivine-nephelinite (nepheline-basalt). The cores represent the necks or feeding-channels of old volcanoes of which the extruded lava and ashes have long since disappeared, and they are usually surrounded by a zone of sandstone of a reddish colour, sometimes with the occurrence of hard white pebbly quartzite formed by the baking and partial recrystallization of the normally friable Sena sandstone during the uprising of the molten rock while the volcanoes were in eruption.

On the left bank of the river opposite Sena, crystalline rocks of the pre-Cambrian age outcrop in a north-easterly direction where they form the southern extremity of the Port Herald hills. They consist of variable granitic and other light-coloured gneisses as well as micaceous and hornblendic gneisses and schists. An outcrop of Karroo basalt takes the form of an irregular strip which is overlain on the western side by the Mutarara rhyolites and sandstones; on the east it is brought down against the crystalline rocks by an

---

[1] The MS. contains a considerable amount of additional information regarding the river and the geological formation of its basin; it may be consulted in the Institution Library.—Sec. Inst. C.E.

important fault which runs slightly west of north; to the south the fault passes beneath the Mutarara beds, which consist of an upper and lower series of reddish pebbly sandstones separated by a group of reddish and mauve rhyolitic lava, ashes and volcanic debris. The sandstones are usually coarse and feldspathic and slightly calcareous. The volcanic groups give rise to the greater part of Mutarara ridge, and from the site of the Lower Zambezi bridge it can be seen in profile half-a-dozen miles to the north, where it forms a high double ridge which consists mainly of thick lava flows tilted towards the river. The group appears to reach a maximum thickness at this northern point, where it comprises about 900 feet of lava with intercalated tuffs, agglomerates and sandstones. It dies out in a southerly direction towards Mutarara and it dies out also towards the north, for near Sinjal the volcanic rocks are again missing from the sequence. The upper sandstones from the western side of Mutarara ridge, the rocky floor beneath the river Zambezi and the country to the west of the river dip at an angle estimated at 4 degrees, and the thickness of the sandstones outcropping between Mutarara ridge on the left bank of the river and Sena hill on the right bank cannot be less than 1,250 feet [1] (*Fig. 2*, p. 330).

The site of the bridge is in the depressions forming the southern end of the Great Rift Valley, and as down-faulting movements which formed the Rift are known to have continued in recent times and movements are still taking place, it was necessary to consider such movements in relation to the bridge. Tremors occur somewhat frequently in the northern part of Nyasaland, and one of them at least is recorded as having extended southwards as far as the bridge area in 1910. They occur less frequently in southern Nyasaland, but one was felt in 1930 over a large area reaching to the site of the bridge. Although these tremors were of no great intensity it was impossible to make a prediction of the intensity and character of future disturbances; it was therefore resolved to take minor earth-movements into consideration in the design of the piers of the bridge as a precautionary measure.

In June 1934, after all the main wells had been founded and more than half the spans had been erected, distinct tremors were felt in the immediate neighbourhood of the bridge but they were not of sufficient intensity to cause any movement of the piers.

The Site of the Bridge.

International navigation rights are in force on the river Zambezi, and it was a condition of the concession by the Portuguese Govern-

[1] Footnote 1, p. 328.

ment that the bridge should permit the passage of river-steamers at all times. This condition entailed either a low-level bridge with an opening span or a high-level bridge with a headway of 27 feet above the highest flood. The former was out of the question owing to the instability of the navigation channels, already referred to, and it was

Fig. 2.

Geological Map.

therefore necessary to design a bridge with the headway required for navigation.

Consideration was first given to the locality of the ferry crossing between Murraça and Chindio, whereby the existing railway connections could be used. This site would not have been out of the question had it been only the necessity of raising the surface line on both sides of the river to above flood level, as a large quantity of earthworks would be involved at any point of crossing. The deciding factors against it were the insecurity of the island of Inyangoma on

which 24 miles of the Central Africa Railway is built from Chindio to Baue, and the unstable nature of the river banks. Other sites

Fig. 3.

PLAN OF SITE.

were considered, but that selected at Sena, 25 miles above Murraça, offered advantages of primary importance (*Fig. 3*).

The Physiography of the River at the Site of the Bridge.

The formation of the country in the neighbourhood of both banks indicated that the sandstone beds extended across the river, and although there is an outcrop of hard sandstone on the left bank well above high-flood level which made that bank secure against erosion and avulsions, there was no visible outcrop on the right bank. Borings indicated, however, a hard formation at sufficiently high levels to make the right bank secure also, and it was therefore considered that the site offered security for both abutments and the approaches to the bridge.

Borings along the selected centre-line of the bridge indicated that the river-bed was composed of transported sands to considerable depths, with thin beds of sandy clay, and in some places black clay, overlying the sandstone beds. The sandstone beds were, however, at sufficiently high levels to provide foundations at a reasonable depth for two-thirds of the total length of the bridge. The remaining one-third of its length was in the central part of the river where the trough formed in the sandstone beds is deepest. Borings in this part of the river through the sand overlying the sandstone reached a maximum depth of 145 feet below the bed of the river, and as there was every probability that the sand extended to considerably greater depths, it was resolved to found the piers in the central part of the river in sand, or in clay should this material be met with at the required founding level.

The main channel, which varies in width from year to year (the limits being between 515 and 1,030 feet at low-river level), flows close to the left bank over hard sandstone for a width of 500 feet from the bank, beyond which point the sandstone is overlain by sandy clays and sand. This channel has existed from time immemorial, and although there is every reason to believe it is stable it could not be considered to be of such a permanent nature as to justify a low-level bridge with an opening span at this point. For a distance of 3,500 feet from the right bank of the river the ground is undulating with an easy slope towards the east, and during a maximum flood it would be covered by water from 5 to 15 feet deep. Vegetation is thick and it is dotted with full-grown thorn trees. The formation consists of thin beds of sandy clays and sand from 15 to 105 feet deep overlaying the sandstone, and a surface of mostly black alluvial deposit to a depth of from 2 to 4 feet. Running through this stretch of ground 2,200 feet from the river-bank, there is a narrow depression known as the old Sena channel which served the Portuguese fort at Sena in 1504, but in which water now flows

only when the river Zambezi is in high flood. There is evidence that 30 years ago small river-steamers with barges navigated this channel, but only as a subsidiary to the main channel on the other side of the river. The next 3,400 feet consists of a bank of sand covered with reeds and grass with two narrow channels running through it which were formed at distances of $\frac{3}{4}$ mile and 3 miles above the site of the bridge. This bank would be covered by from 7 to 20 feet of water during a maximum flood, and it is of a less stable nature than the stretch of ground above referred to. At the end of this bank there is a subsidiary channel 500 feet wide, and the remaining width of the river-bed consists of low banks formed by shifting sand which change their position yearly according to the vagaries of the currents above the bridge. They are exposed to heights of from 1 to 10 feet at low-river level.

The deepest part of the river, including the main channel, is over a width of 5,000 feet on the east side; although its permanency on that side is not assured, the possibility of this maximum section of discharge being diverted to the extreme west side was considered to be very remote, and this, coupled with the favourable nature of the ground on the west side of the river, permitted a lighter construction of the bridge, with well-secured foundations, for a length of 3,002 feet 10 inches at the west end, thus considerably reducing the total cost.

The depth of the foundations in sand for the piers of the main spans was determined in the following manner during the survey and preparation of the project. There was some difficulty in obtaining any data for maximum depths of scour at the site of the bridge owing to the irregularity of the annual rainfall in the catchment area. In 1928, the year of the survey, the river rose no more than 10 feet in the rainy season, as compared with a maximum rise of 25 feet from a low-water level of R.L. 120·00 to a maximum high-flood level of R.L. 145·00 at the site of the bridge. The survey was therefore continued in 1929, and on 9 February the river rose to R.L. 139·80 at the site of the bridge, or a rise of 19·80 feet. Observation at the site of the bridge the previous year had shown that no great scour was to be expected there owing to the comparatively straight run of the river along the left bank, which consisted of hard sandstone dipping into the river at an easy slope. As experience of changes of course both above and below the site of the bridge showed that the conditions at the site were liable to change, it had been decided to ascertain the greatest depth of scour occurring on the length of river extending from 15 miles below to 15 miles above, as well as at the site of the bridge. Soundings were therefore continued at three points from 9 February, and on 27 March,

following a flood lasting 6 days from 16 to 21 March at R.L. 135 at the site of the bridge, a maximum sounding of 60 feet below low-water level was obtained at an eroding bend in the river, at Casquete, 10 miles below the site of the bridge. As the greatest depths of scour in alluvial rivers appear to occur, not during the highest floods when the flood is passing straight down the river, but on a falling river when the channels are taking excessive curvature, it was assumed that 60 feet was about the maximum scour that would occur at the site of the bridge when the straight flow had given place to a winding channel. The river at the site of the bridge being 2¼ miles wide in high flood, it was not difficult to foresee the possibility of the occurrence of a bend-channel sooner or later at the bridge. The deepest scour is, however, to be expected where the river is cutting a hard bank, but as at the site of the bridge the river-bed is of clean sand and the left bank which is formed of hard sandstone is of a shelving character, no such scour is likely to occur as might be the case if the bank were steep. Assuming that a foundation 50 feet below bed-level would be safe, it was therefore considered that a depth of 110 feet below low-water level would be sufficient, the local scour due to obstruction of the piers being taken care of by surrounding each pier with a ring of heavy pitching stone, which experience shows will sink as scour at the pier takes place, without much movement downstream. The greatest depth of scour below low water observed in 1929 at the bridge site was 31 feet on 26 March. The greatest depth of scour observed since the piers have been constructed was 28 feet in the main channel following a flood which reached R.L. 137·62.

The Hydrography of the River at the Site of the Bridge.

The width and depth of the river are as follows :—

Width at maximum-flood level . . .	R.L. 145·00	12,200	feet.
„ „ lowest recorded river-level .	„ 119·80	515	„
Maximum depth at maximum-flood level		53	„
„ „ low-river level .		28	„

The highest flood-peaks during the last 100 years ranged from R.L. 145·00 (a rise of 25 feet above low-water level) in 1840, to R.L. 131·50 (11·50 feet rise) in 1930. The average annual rainfall at the site of the bridge is 25 inches, and the sectional area of discharge for which the bridge is designed is 195,300 square feet. The fall of the river is shown in *Fig. 4*. The fall was observed between the centre line of the bridge and a point 1·86 mile below the bridge. From this

point to Murraça the rate of fall decreases considerably. Based on observations made at lower flood-levels, it is estimated to be 1·50 foot per mile for a distance of 21·55 miles, with a flood-level of R.L. 145. It will be seen from *Fig. 5* that the velocities calculated by Man-

Fig. 4.

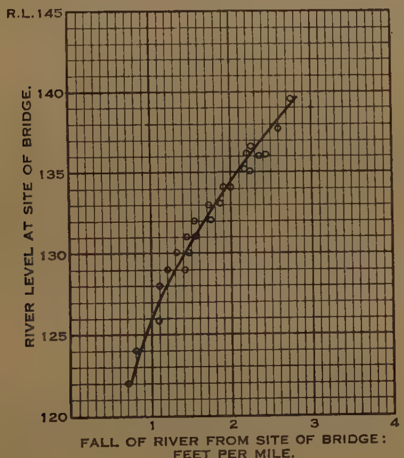
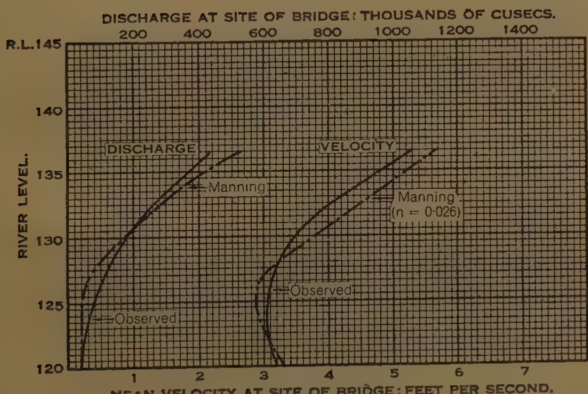


Fig. 5.



ning's formula [1] with $n = 0{\cdot}026$ closely approximate to the observed velocities. It is not possible to give the velocities and discharges during very high floods, as the river only rose to moderate levels during the period it was under observation by the Author, but it

[1] H. W. King, "Handbook of Hydraulics," p. 182. Second Edition, New York, 1929.

is probable that with a maximum flood to R.L. 145 the velocity would be at least 7·5 feet per second with a discharge of 1,500,000 cusecs.

The Bridge.

Length.—At the maximum-flood level of R.L. 145 the bridge will be over water for its entire length. It consists (Fig. 6, Plate 1) of the following component parts :—

Viaduct at west end	1,847·83 feet.
Seven secondary spans of 165·00 feet	1,155·00 ,,
Thirty-three main spans of 262·50 feet	8,662·50 ,,
Six approach spans at the east end of 66·50 feet . .	399·00 ,,
Total	12,064·33 feet.

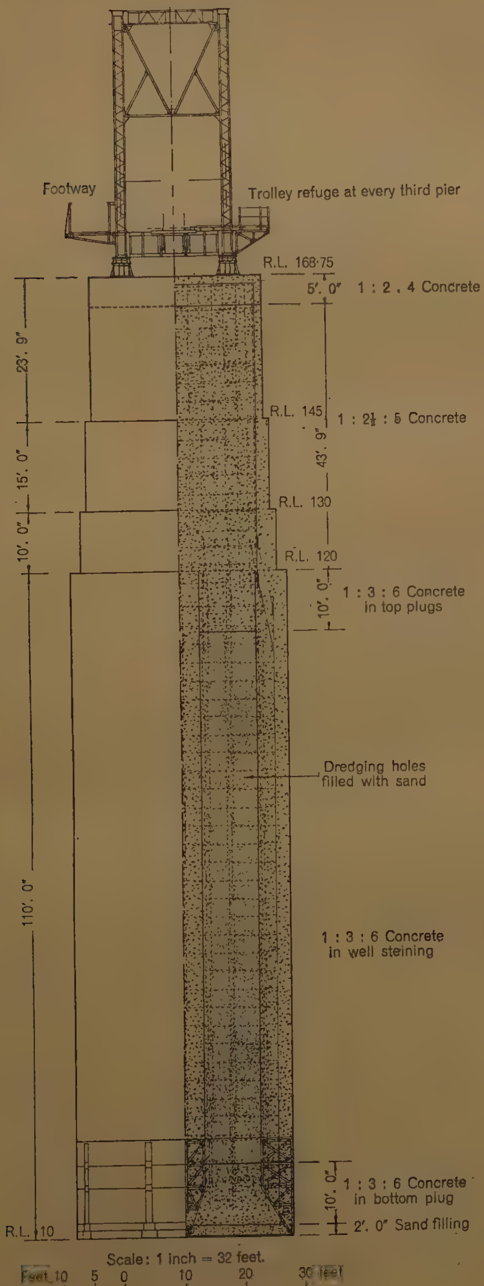
It is shown in comparison with other bridges of exceptional length in Fig. 7, Plate 1.

Main Features of the Design.

The bridge is designed for a single-track railway of 3 feet 6 inches gauge, the clear width being 14 feet 6 inches and the headway 27 feet above maximum-flood level, for navigation of a width of river of 3,937 feet on the east side. The main part of the bridge consists of steel through spans supported by concrete piers built on wells. A footway is provided on the upstream side of the bridge and trolley refuges at every third span. The bridge is straight throughout its length. At the west end the rail-level is at R.L. 155 and the track is level for a length of 3,002·83 feet. For the next 4,983 feet there is a gradient from R.L. 155 to R.L. 177, and the remaining length of 4,078·50 feet is level.

Main Wells and Well-Curbs.—The wells are rectangular in plan with semicircular ends, and are 36 feet long by 20 feet wide. They are of the open-dredging type with two dredging-holes 9 feet 6 inches in diameter. The steining of 1-to-3-to-6 concrete, placed in situ, is 5 feet thick, and is reinforced as a provision against earthquakes (*Fig. 8*). The well-curbs for founding in sand or soft material were 8 feet 4½ inches in height and were built up of ⅜-inch steel plates (Figs. 9, Plate 1). The cutting-edge consisted of a ⅝-inch plate and two 4-inch by 4-inch by ½-inch angles riveted to the curb-plates and strengthened against lateral stresses by ⅜-inch and ⅝-inch cover plates (Figs. 10, Plate 1). The well-curbs for founding in rock were 8 feet 1½ inch in height and were of similar design except that the cutting-edge was flat. The cutting-edge consisted of ¾-inch and

Fig. 8.



MAIN WELL AND PIER.

$\frac{5}{8}$-inch overlapping bent plates and a 4-inch by 4-inch by $\frac{1}{2}$-inch angle riveted to the curb plates and stiffeners (Figs. 11, Plate 1). Strakes were added to the curb in sections 3 feet $9\frac{1}{8}$ inches high to the height required for pitching the curb in water. The inside lining, or upper trunk, above the top of the curb was built up of $\frac{3}{8}$-inch plates in sections of the same height as the strakes, and the whole formed a water-tight caisson. The minimum form of construction was for curbs which were founded on a sandbank, and consisted of a curb with an incomplete strake. The angle which the cone-plate of the curb formed with the vertical was 32 degrees 20 minutes, and as this angle had an important bearing on the effort required to sink wells it will be referred to later. The weight of a curb for founding in sand was 39 tons and that for founding in rock weighed 37 tons. The weight of an incomplete strake was 7 tons and of a complete strake 12 tons.

For sinking under compressed air a cast-steel dome with a hinged door (Figs. 12 and 13, Plate 2) was seated on a cast-steel ring $\frac{7}{8}$ inch thick cast to an angle of 45 degrees and riveted to the plate of the first section of inner trunk above the curb; the joint between the dome and the seating-ring was made water-tight by means of a continuous indiarubber ring, which was fitted to the dome. Another steel ring was fitted to the inner trunk 11 inches above the seating-ring for clipping on the lever-latches of the dome. Shafting 4 feet in diameter connected the dome to the air-lock at the top of the well.

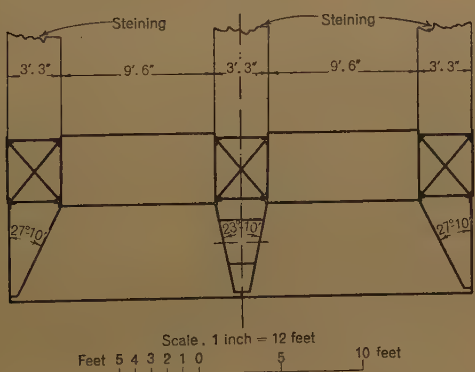
This apparatus was designed by Mr. H. J. Fereday, M. Inst. C.E., for sinking the wells for the Kalabagh bridge over the river Indus. The essence of the design is that air-compression can be applied to any well at any time and that the domes can be removed for continuing sinking by open dredging. Its use for well-sinking for bridges in India has resulted in great saving of time, and the Author can testify as to its successful employment in connection with the wells for the Lower Zambezi bridge.

The holes dredged below the cutting-edge of wells founded in sand were back-filled with sand to a height of 2 feet above the cutting-edge, and the bottom of the well was plugged with 1-to-3-to-6 concrete to a height of 10 feet above the sand filling, the concrete being lowered through the water in hopper-skips. The dredging holes were then filled with sand to within 10 feet of the top and were plugged with 1-to-3-to-6 concrete. The holes dredged below the cutting-edge of wells founded in clay were sometimes as much as 4 feet 6 inches deep. These holes were filled with concrete in a continuous operation with the placing of the bottom plugs. In the case of wells founded on rock under compressed air, the bottom was levelled off and as much as possible of the concrete for the bottom

plugs was placed in the air chamber under pressure, leaving sufficient clearance for closing the door of the dome. Pipes 3 inches in diameter were placed at intervals of 7 feet 6 inches around the air-chamber, with one end close to the cutting-edge and the other end above the level to which the concrete was to be placed. The object of these pipes was to relieve the hydrostatic pressure on the concrete as the air-pressure was gradually reduced to zero. Quick-setting cement was used for the concrete, which was allowed 24 hours to set under the full air-pressure. The pressure was then gradually released during a period of 12 hours. After the removal of the dome the plug was completed with concrete placed in the water.

Secondary Wells and Well-Curbs.—The wells and curbs for the 165-foot spans were designed on lines similar to those for the main

Fig. 14.



spans, with the important exception that the angle between the cone-plate of the curb and the vertical was 27 degrees 10 minutes at the outer cutting-edge and 11 degrees 35 minutes at the central cutting edge (*Fig. 14*), as compared with 32 degrees 20 minutes and 20 degrees 10 minutes respectively for the main wells (*Fig. 15*). The resistance to sinking by the ground around the cone-plates was correspondingly reduced by this alteration. The wells are 28 feet 9 inches long by 16 feet wide and the steining is 3 feet 3 inches thick. The dredging-holes are of the same diameter as for the main wells, and similar provision is made for sinking under compressed air.

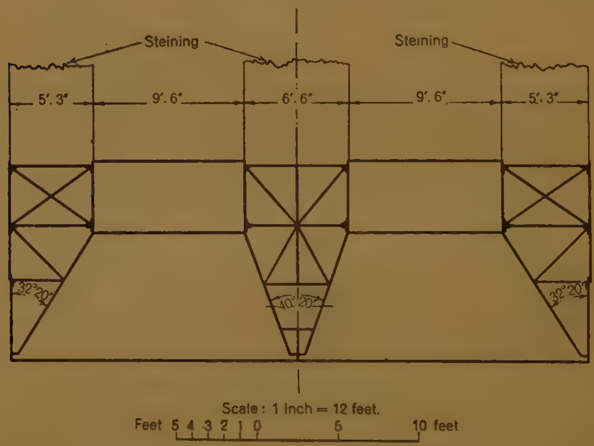
Piers.—The piers throughout the bridge are rectangular in plan with semicircular ends. They have vertical sides and are stepped at intervals to bring them to the required dimensions at the top. The main piers are founded on the wells at R.L. 120 and the secondary

piers at R.L. 130. Main piers Nos. 33 and 34 and the piers for the approach-spans at the east end of the bridge are founded directly in sandstone rock. The piers are built of 1-to-2½-to-5 concrete and capped with 1-to-2-to-4 reinforced concrete 5 feet deep. As in the case of the wells, the concrete of the piers is reinforced as a provision against earthquakes.

Well-Sinking.

Wells sunk entirely in sand or partly in sand and partly in soft clays were required to be founded at a depth of 110 feet below low-

Fig. 15.



river level (R.L. 10), which is equivalent to 50 feet below the level of the estimated maximum scour of the river-bed. Where rock was met with at higher levels the wells were required to be founded as follows:—

For rock at 80 feet or less below low-river level (R.L. 40), the wells to be sunk 10 feet into the rock.

For rock at 100 feet below low-river level (R.L. 20), the surface to be levelled off for founding the well.

For rock between 80 and 100 feet below low-river level, the well to be sunk into the rock to a proportionate depth between the foregoing limits.

The specified limit of depth for sinking wells under compressed air was 100 feet below low-river level (R.L. 20).

The sieve analysis of an average sample of the sand in the river-bed was as follows :—

Size of grains: inch.	Analysis: per cent.
Over $\frac{1}{4}$	1·10
$\frac{1}{4}$	2·50
$\frac{1}{8}$	8·40
$\frac{1}{32}$	22·80
$\frac{1}{50}$	29·00
Under $\frac{1}{50}$	36·20
	100·00

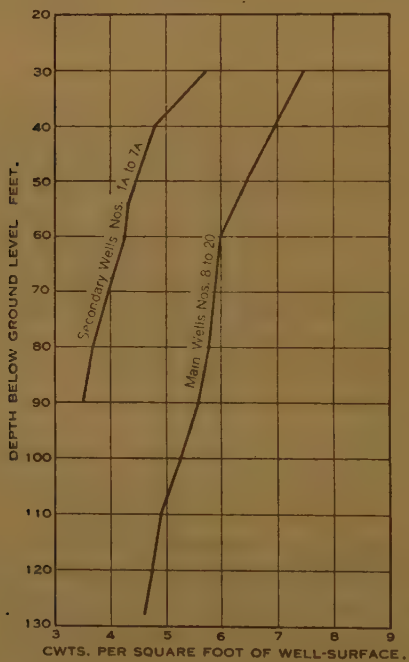
The sandy clay is of a reddish to brown colour with a low degree of plasticity. It is hard when dry but small pieces disintegrate when placed in water. Strata or pockets of black clay were met with beneath considerable depths of sand and sometimes beneath the sandy clay. The black clay is very hard when dry but becomes plastic when saturated with water. The sandstone is reddish in colour and coarse-grained. On the east side of the river the beds are hard, but towards the west the aggregation of the mineral grains is loosely cemented together and the rock is of a softer nature. Compression-tests made on the sandstone gave failures ranging from 174 to 61 tons per square foot when dry and from 139 to 26 tons per square foot when damp. Failure followed closely upon the appearance of the first cracks in the cubes tested. The cubes were prepared from blocks of sandstone taken from the wells at or near foundation-level.

The ground generally was favourable for accurate sinking and no major difficulties were met with at any of the wells. The following were the maximum settlements of the main and secondary piers in the different materials in which they were founded. The observations were made after trains weighing about one-half of the live load for which the bridge was designed had been running over it for 3 months :—

	Main wells: inch.	Secondary wells: inch.
Founded on sand	$\frac{25}{32}$	—
,, ,, clay	$\frac{21}{32}$	—
,, ,, soft sandstone	$\frac{7}{16}$	$\frac{3}{16}$
,, ,, hard ,,	$\frac{7}{32}$	—

Sinking-Effort.—The average sinking-effort of main wells Nos. 8 to 20, which were sunk entirely in sand without kentledge or pumping, was 4·7 cwts. per square foot, as shown by the composite graph (*Fig. 16*). The sinking-effort of the secondary wells Nos. 1A to 7A in sand is also shown on this graph for the purpose of comparison, and will be referred to later. No. 11 was sunk 128·16 feet, this being the greatest depth to which any of the wells were sunk entirely in sand, and at this depth it had reached the required

Fig. 16.



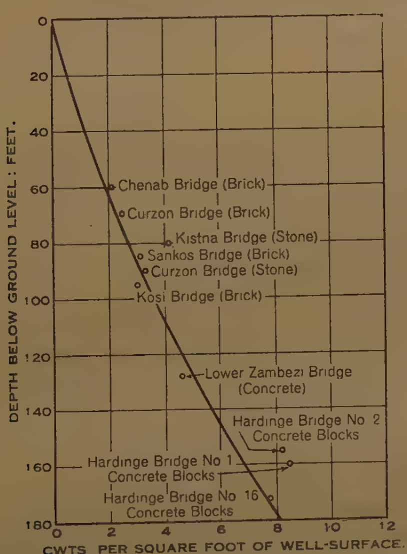
founding-level. The ground-level was R.L. 138·10, which was 18·10 feet above the required level for the top of the well. The river-level at the time the well was founded was R.L. 128·46 and the required level for the top of the well was therefore 8·46 feet below water-level. The practice adopted in these circumstances was to construct a short length of pier on the well, not exceeding 10 feet, in order to keep the concrete above water level during the final stages of sinking. It was found that the change in the position of the wells during the final 10 feet of sinking was negligible, and any small differences in level were corrected with the subsequent

lift of the concrete of the pier. The total weight of well No. 11 was as follows :—

Curb	46 tons.
Well steining	3,683 ,,
Pier (8·75 feet)	205 ,,
	3,934 ,,
Less buoyancy	1,530 ,,
Total	2,404 tons.

which produced a sinking-effort equivalent to 4·6 cwts. per square foot of well surface.

Fig. 17.



SINKING-EFFORT FOR DIFFERENT BRIDGES.

When a well has been sunk as far as it will go without loading kentledge and without pumping, it reaches what the Author will hereafter term its "critical depth." The final stages of sinking No. 11 presented no difficulty; although it was evident it had not reached the critical depth when founded, the Author is of the opinion that it was very near it, and in applying the corresponding sinking-effort when dealing with skin-friction under the next heading, the Author has assumed that the depth to which this well was sunk was its critical depth. The graph (*Fig. 17*) has been prepared from

information given by Sir Robert Gales, M. Inst. C.E., of sinking wells in India.[1] Well No. 11 has been added to the graph as representing the wells which were sunk entirely in sand for the Lower Zambezi Bridge. The points on the graph are the critical depths to which the various wells were sunk entirely in sand with the corresponding sinking-efforts. Sir Robert Gales's experience was that too much stress cannot be laid on the importance of providing the weight in the steining itself instead of in the form of artificial loading,[2] and no doubt the great critical depths to which the Hardinge bridge wells were sunk were to a very great extent due to improved design, which was still more apparent in the case of the Lower Zambezi bridge wells. In this connection the ratio of the area of the horizontal section to the perimeter of the wells, coupled with the weight of the material with which the wells are constructed, is of importance; the following is a comparison of these figures :—

	Area/perimeter: ratio.	Concrete: lbs. per cubic foot.	Greatest critical depth sunk: feet.
Hardinge bridge wells	8·91	158	172
Lower Zambezi bridge main wells .	5·19	157	128

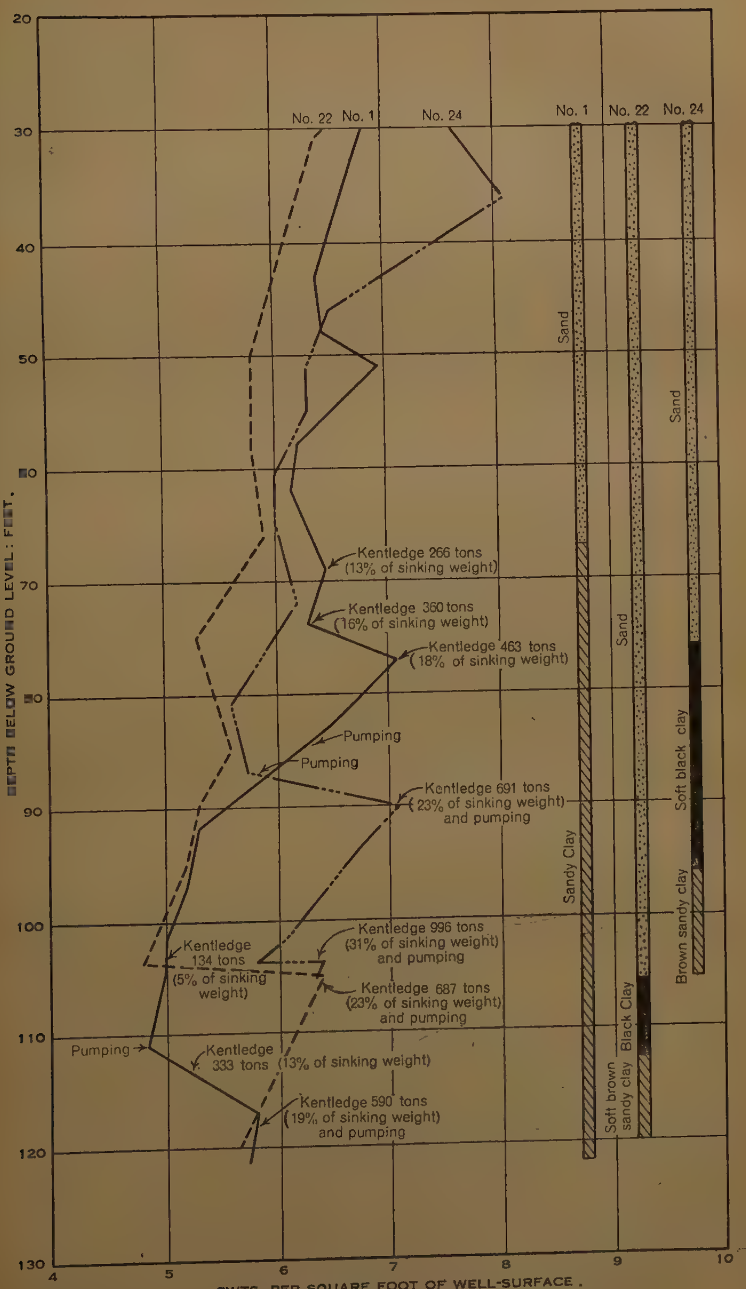
The heavy weight of the concrete is accounted for by basalt stone with a high specific gravity having been used for the coarse aggregate for both bridges.

Fig. 18 shows the sinking-efforts for main wells Nos. 1, 22 and 24. No. 1 reached the bed of sandy clay at a depth of 67 feet, and when it had entered 2 feet it was necessary to load kentledge. The well was sunk in sandy clay to the required depth with kentledge equivalent to 19 per cent. of its sinking-weight and the assistance of pumping, with a sinking-effort of 5·7 cwts. per square foot. No. 22 reached black clay at a depth of 106 feet, when kentledge equivalent to 23 per cent. of its sinking-weight was loaded, and, with the assistance of pumping, the well was sunk in 7 feet of black clay and 7 feet of sandy clay to the required depth with a sinking-effort of 5·7 cwts. per square foot. No. 24 reached soft black clay at a depth of 76 feet, and when it had been sunk 14 feet into this material with the assistance of pumping, kentledge was loaded equivalent to 23 per cent. of its sinking-weight. The well was sunk a further 6 feet in

[1] "The Hardinge Bridge over the Lower Ganges at Sara." Minutes of Proceedings Inst. C.E., vol. ccv (1917–18, Part I), p. 33.

[2] "The Curzon Bridge at Allahabad." Minutes of Proceedings Inst. C.E., vol. clxxiv (1907–8, Part IV), p. 27.

Fig. 18.



SINKING-EFFORTS FOR MAIN WELLS Nos. 1, 22, and 24.

the black clay and 9 feet in sandy clay, and was founded at R.L. 13·97 with kentledge equivalent to 31 per cent. of its sinking-weight, as well as pumping, with a sinking-effort of 6·4 cwts. per square foot.[1]

Skin-Friction.—The depth to which it is necessary to sink wells in sand, and to found them on sand, is governed by the known or probable maximum depth of scour of the river-bed at the site of the bridge, and as a foundation of sand does not generally offer the resistance necessary to carry the full load safely without the assistance of skin-friction, the measure of the skin-friction is an important element in the design of bridge-wells.

In main wells Nos. 8 to 20, which were sunk and founded in sand, the bottom of the dredging-hole was generally 2 feet below the level of the cutting-edge of the well-curb. As no pumping was done in these wells the sand would be in a compressed condition beneath the cutting-edge, offering resistance to sinking in addition to the resistance by skin-friction, so that the sinking-effort cannot be equated to the resistance by skin-friction at the critical depth; the latter is less by the resistance beneath the cutting-edge. The actual measure of this resistance is not calculable, but taking it as 6 tons per square foot acting beneath the cutting-edges of the vertical plates and the horizontal part of the cone-plates of the outer and central cutting-edges, the resistance is 438 tons; this is equivalent to 0·84 cwt. per square foot of resistance by skin-friction, and makes the skin-friction 3·76 cwt. per square foot on main wells Nos. 8 to 20.

In the case of wells sunk by open dredging in clay, there is resistance by the wall of clay around the cone-plates above the cutting-edge, in addition to the resistance beneath the cutting-edge. Those wells which were founded in clay were inspected by a diver, and it was reported that the walls of clay were from 3 to 4 feet in height and from 4 to 5 feet wide at the base. The removal of this wall of clay is one of the problems of well-sinking by open dredging. Attempts were made to dislodge it by explosives, by air-jets and by a heavy workmanlike tool designed by the contractors, but the attempts only met with partial success. Owing to the varying and indeterminable measure of resistance around the cone-plates of the well-curbs when sunk in clay by open dredging it was not possible to arrive at the measure of the resistance due to skin-friction alone. Nor was there any well sunk entirely in clay under compressed air with the cutting-edge free from resistance, from which the resistance by skin-friction could be arrived at.

Opinions differ as to whether the resistance by skin-friction in-

[1] The MS. contains further information on sinking effort; it may be seen in the Institution Library.—SEC. INST. C.E.

creases directly with the depth or in greater ratio. Although it may appear from the curve of sinking-efforts at critical depths (*Fig. 17*) that it increases in greater ratio, the comparison of the minimum sinking-efforts of wells of different design and constructed with materials of different weight does not perhaps offer sufficient evidence. So far as can be deduced from the sinking-efforts of the Lower Zambezi bridge wells, the Author is of the opinion that the resistance by skin-friction increases directly with the depth in material of a homogeneous nature.

Load on Foundations.

The weight of a main well sunk to R.L. 10 with a pier of maximum height is 3,904 tons, which with the dead and live loads amounting to 1,278½ tons gives a total weight of 5,182½ tons. The following Table gives a comparison of the foundation-pressures in tons per square foot with those for the Willingdon and Hardinge Bridges in India :—

Description.	Lower Zambezi bridge.	Willingdon bridge.[1]	Hardinge bridge.[2]
Pressure on base of wells after deducting buoyancy at low-river level	8·17	5·94	9·00
Reduction due to skin-friction . .	1·46	2·40	—
Net pressure	6·71	3·54	—
Skin-friction allowed: cwts. per square foot	3·76	5·00	—
Nature of foundation	Compacted sand.	Yellow clay and sand.	Compacted sand.

In the case of the Lower Zambezi bridge the reduction due to skin-friction has been calculated on the surface area of the well from the founding-level of R.L. 10 to R.L. 60, the level of the estimated maximum erosion of the river-bed.

Steelwork.

Main Spans.—The thirty-three main spans consist of trusses of the sub-divided Pratt type with polygonal top chords (Figs. 19 and 20, Plate 2). They are 258 feet in length between centres of bearings and 38 feet $7\frac{15}{32}$ inches deep between the neutral axes of the booms.

1 Robert Mair, "The Willingdon Bridge, Calcutta." Minutes of Proceedings Inst. C.E., vol. 235 (1932–33, Part I), p. 36.

2 Footnote 1, p. 344.

The spans are divided into eight main bays of 32 feet 3 inches, each bay being sub-divided by sub-verticals into two sub-bays each 16 feet 1½ inches; this is the distance between the cross girders. Each span weighs 320 tons exclusive of the bearings and footway. The knuckle-bearings are of cast steel with pins 6 inches in diameter, and at the expansion end they rest on 4-inch diameter rollers, ten to each bearing.

The spans are designed to the Uganda Railway "1925 scale" Table of equivalent loads, reduced proportionately to provide for locomotives with an axle-load of 16 tons. The loading used is equivalent to a train weighing 1·75 ton per foot run drawn by two 4–8–2 locomotives in tandem each weighing 135 tons complete with tender.

The load on the bearings is as follows:—

Weight of span and bearings	334 tons.
Weight of track	26 ,,
Total per span	360 tons.
Total per bearing	90 tons.
Equivalent dead load from footway	39 ,,
Total dead load per bearing	129 ,,
Live load per bearing	133 ,,
Impact ,, ,,	71
	333 tons.

Bearing pressure = 19·4 tons per square foot.

The specification provided for all similar parts of the steelwork to be made interchangeable by the use of steel-bushed jigs, and for the first span to be completely erected in the contractors' yards for the purpose of checking the accuracy of the templates. It also provided for further spans to be erected as required, built from parts selected at random, in order to check the accuracy of the work. Three main spans were completely erected in the contractor's yards in accordance with the foregoing provisions. During the erection on site the manufacture and accuracy of the steelwork was found to be of a high standard. The convenience of the interchangeability of the parts both during manufacture and during the erection of a large number of spans on site cannot be over-emphasized. The main spans were designed with an erection camber of 2¼ inches. This camber was set with 20-ton screw-jacks, one to each panel and sub-panel point.

Secondary Spans.—The seven secondary spans consist of trusses of the sub-divided Pratt type with parallel chords (*Figs. 21* and *22*, pp. 350, 351). They are 161 feet 3 inches between centres of bearings and 27 feet deep between the neutral axes of the booms. Each span weighs 169 tons exclusive of the bearings and footway. The spans were designed with an erection camber of 1 inch.

Approach-Spans.—The six approach-spans at the east end of the bridge are girders of the whole-plate deck type. They are 65 feet 10 inches long and each span weighs 24 tons exclusive of the bearings and footway.

VIADUCT.

The viaduct at the west end of the bridge consists of whole-plate deck-girder spans supported by steel trestles of two "Phœnix" columns of $9\frac{7}{8}$ inches internal diameter and $\frac{1}{2}$ inch thick. The fifty-five trestles are arranged in series of eight, seven spaced at 31 feet 10 inches centres and the other at 39 feet 8 inches. The columns are braced laterally by 6-inch by $3\frac{1}{2}$-inch by 16·48-lb. channels, and every third and fourth trestle is braced longitudinally by two 15-inch by 4-inch by 36·37-lb. channels with battens and lacing bars (Figs. 23, Plate 2).

The trestles are founded on groups of reinforced "Vibro" piles capped with reinforced-concrete slabs 3 feet 6 inches thick. The piles are 17 inches in diameter and are made of 1-to-2-to-4 concrete with "Ferrocrete" cement. The single trestles are supported by a group of six piles and the double ones (*Fig. 24*) by two groups of eight, the caps of the latter being tied together by reinforced-concrete beams.

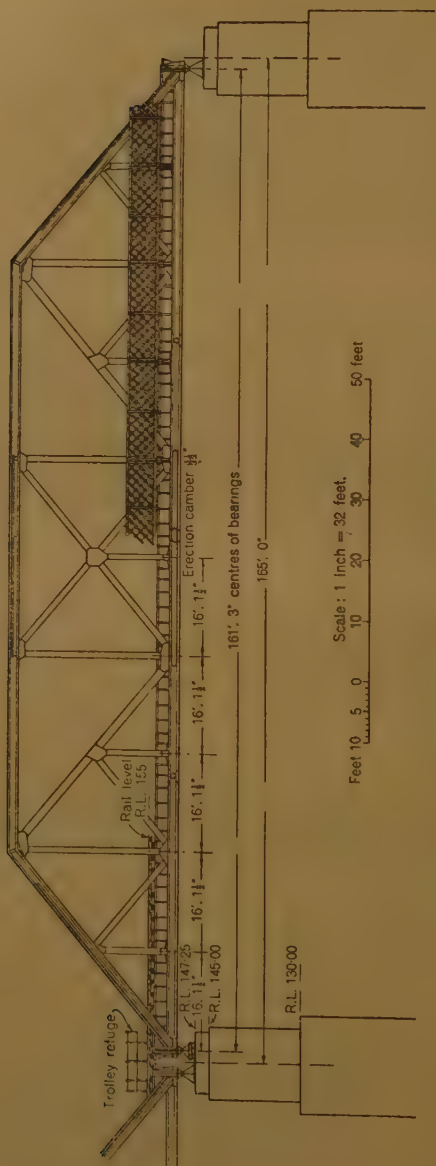
The specification called for the 16-inch steel pile-tube to be driven until a driving resistance of 84 tons had been reached, as indicated by a final set of 2·5 inches for ten 3-foot 6-inch blows of a 2-ton single-acting hammer.

The following formula was used :—

$$R = \frac{Wh\eta}{s + \frac{c}{2}},$$

where W denotes the weight of the hammer in tons ;
h ,, height of the fall in inches ;
η ,, efficiency of the blow (45 per cent.) ;
s ,, set per blow in inches ;
c ,, temporary compression of the pile and the ground (0·4 inch) ;
R ,, resistance of the pile-tube in tons.

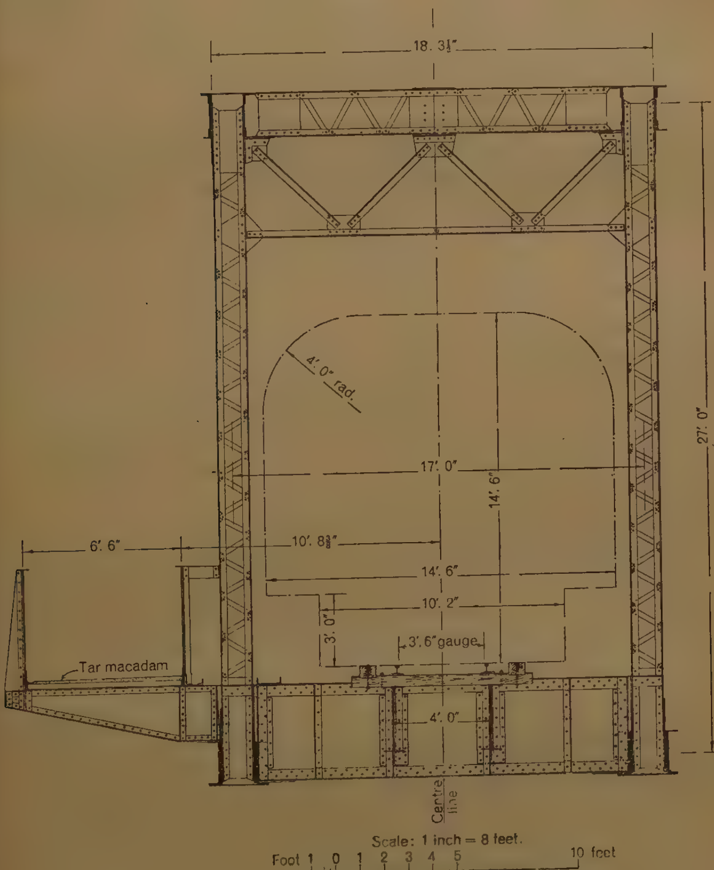
Fig. 21.



SECONDARY SPAN: ELEVATION.

To $R = 84$ tons, 30 per cent. was added as representing the additional resistance by skin-friction of a concrete pile with a corrugated surface, and the resultant resistance $R_1 = 109$ tons was the required bearing resistance of the concrete pile. The working load

Fig. 22.



SECONDARY SPAN: CROSS SECTION.

is 40 tons, so that the factor of safety provided for is 2·73. The helmet consisted of a steel casting fitted with a hardwood driving-cap through which the hammer-blow was transmitted to the pile-tube.

The specification provided that the piles should be generally 40 feet in length, but that if the ground were sufficiently hard to give

a set through the final 4 feet of driving of 0·15 inch per blow the pile-tube should be driven to depths of less than 40 feet. It also provided that the pile-tube should not be driven more than 45 feet unless the set per blow was more than 0·25 inch, in which case it should be driven to the limit of its effective length and additional piles provided to give the required bearing power. In order to avoid damage by vibration and disturbance of the ground it was also required that no successive pile should be driven at a less distance than 5 feet centre to centre from the preceding pile unless the latter had set for 24 hours.

Three hundred and ten piles were driven in lengths up to 40 feet, and ninety piles in lengths from 40 to 53 feet. At the west end of the viaduct hard ground was reached at a higher level than antici-

Fig. 24.

GROUPING OF PILES FOR DOUBLE TRESTLE.

pated, and it proved to be so hard in some places that to drive the pile-tube 4 feet into it was too severe on the pile-driving gear. It was, however, necessary to prove the ground, and it was decided that driving should be continued for 2 feet when a penetration of 0·10 inch per blow had been obtained, but that if this set decreased during the final 2 feet the pile-tube should be driven until a set of 0·05 inch was obtained.

Provision was made for testing piles with a load of 80 tons, which is double the working load, with the condition that the elastic yield of the pile, together with the compression of the subsoil, should not exceed ½ inch and that the settlement should not increase during a period of 48 hours. The tests given in Table I were carried out by means of casting a reinforced-concrete platform on to the head of the pile and loading it with 4-ton kentledge blocks.

TABLE I.

Length of pile: feet.	Final set: inch.	Value of R: tons.	Test load: tons.	Elastic yield and settlement: inch.	Recovery with load off: inch.	Settlement: inch.
40·00	0·29	93	99	0·204	0·120	0·084
39·60	0·26	95	129	0·264	0·096	0·168
46·00	0·18	114	80	0·125	0·062	0·063
28·92	0·06	200	80	0·203	0·109	0·094

The first two piles mentioned in Table I were driven and tested for experimental purposes before the construction of the viaduct was begun, and were in sand for their whole length. The other two piles were tested in accordance with the terms of the specification after having been driven in their proper position for the viaduct foundations, and were in sand with the exception of the last 2 feet of driving, which was in the stratum of soft sandstone. Observations made on the viaduct 2½ years after the driving of the piles was finished and after traffic had been running over the line for several months, showed settlement of the piled foundations of from 0·024 inch to 0·360 inch.

FOOTWAY.

The footway consists of lattice-girders 5 feet high, spaced at 6 feet 6 inches between centres and supported by cantilever brackets, spaced at 16 feet 1½ inches between centres and attached to the outside of the upstream trusses of the main and secondary spans and to the upstream "Phœnix" columns of the viaduct. The floor consists of pressed-steel troughing with a 4-inch span, 2½ inches deep and $\frac{3}{16}$ inch thickness of plate. The paving is of tar-macadam 4½ inches thick with a fall of ½ inch to a drain running close to the outside girder with outlet pipes at convenient intervals. It was required that the viscosity of the tar and the proportioning of the materials should be such as to produce a paving sufficiently elastic to withstand the expansion and contraction of the steelwork, and that it should be free from "tackiness" under the maximum temperature of the locality, which is 110° F. in the shade. Some difficulty was experienced in arriving at a mixture which would meet these requirements, but after having made experiments with various materials in varying proportions the following specification was found to be satisfactory:—

(*a*) Bottom layer . . ¾-inch screened stone mixed with "Shelmac" and consolidated by punning.

(*b*) Top layer . . .	A finished thickness of 1 inch of two parts of $\frac{3}{4}$-inch screened stone to one part of $\frac{1}{8}$-inch screened chips mixed with "Shelmac," consolidated and rolled to an even surface.
(*c*) Mix	$6\frac{3}{4}$ gallons of "Shelmac" heated to 180° F. to 1 cubic yard of stone.
(*d*) Top dressing . .	The surface sealed with "Colas" and dressed with coarse sand or stone chips.

It was found in the course of the experiments that if the quantity of "Shelmac" per cubic yard of stone was kept down to the minimum required for binding purposes, a surface free from "tackiness" was obtained. Basalt stone was used and proved to be more effective than hard sandstone, which was too absorbent and did not bind well.

Railway Track.

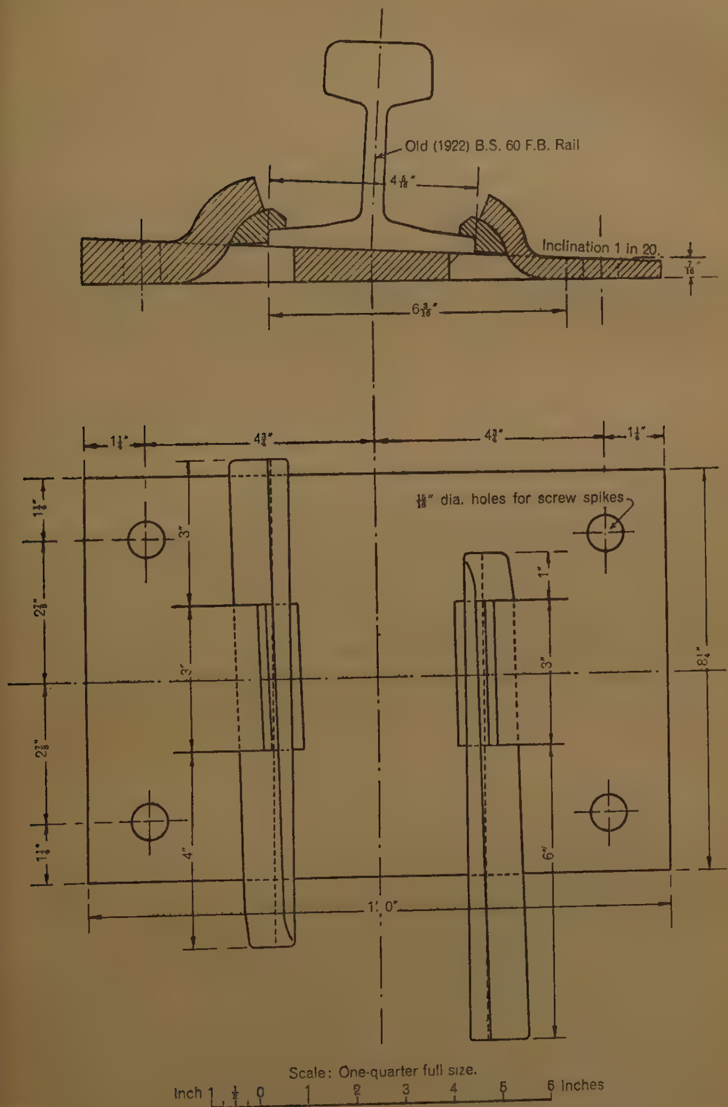
The 3-foot 6-inch gauge single track consists of rails of the British Standard old (1922) 60-lb. flat-bottomed section rolled to special lengths and laid to the camber of the spans, with parallel joints, on steel bearing-plates, which give the rails an inclination of 1 in 20. The fish-plates are of the 4-holed shallow type. The rails are spiked to native hardwood sleepers 7 feet 6 inches long, 10 inches wide and 6 inches thick, spaced at approximately 2-foot centres. An 8-inch by 6-inch outer timber guard-rail is notched to the sleepers and fastened to them with bolts. Anti-creep plates (*Figs. 25*) are laid under the rails and fastened to the sleepers by coach-screws for two rail-lengths in the centre of each main span, for one rail-length in the centre of each secondary span, for two rail-lengths over the third approach span, for one rail-length over each braced bay of the viaduct and for eight rail-lengths of track beyond each end of the bridge. No special arrangement is made for expansion of the rails other than the customary allowances at the joints.

Concrete.

The quantity of concrete used for the construction of the bridge was 96,400 cubic yards. The coarse aggregate consisted of basalt rock with a specific gravity of 3·1, broken to the required sizes and screened. The sand weighed 98 pounds per cubic foot measured loose and dry. The cement was manufactured in Great Britain and conformed to the British Standard Specification. The coarse aggregate and the sand were gauged by loose measure and the cement by weight at 90 pounds per cubic foot. The coarse aggregate was sprinkled with water before use during the hot season.

Basic proportions such as 1 to $2\frac{1}{2}$ to 5 and 1 to 3 to 6 were

Figs. 25.



ANTI-CREEP PLATES.

specified for the concrete for the various parts of the work, but provision was made that the proportions should be adjusted where

necessary so that the proportion of mortar should be 10 per cent. in excess of the voids in the coarse aggregate. These voids varied according to the quarry from which the stone was obtained. The voids in the sand averaged 38 per cent.

Field compression-tests were made of the concrete in cubes 6 inches square. The moulds were filled with concrete taken as delivered *in situ*, and tamped in the moulds in 2-inch layers with twenty-five strokes of a standard steel tamper 15 inches long, with a tamping-face area of 1 square inch and weighing 4 lbs. The moulds were buried in damp sand for 24 hours before removal to the testing room and were kept damp until tested. Slump tests were made of the concrete immediately before placing it in the moulds. The facework of the concrete was good. The average weight of the concrete was 157 lbs. per cubic foot. The sand was specified to be measured dry, but it was obtained from the bed of the river and was in a damp condition; tests showed that the bulking by dampness was as high as 28 per cent. The quantity of sand gauged was increased according to its degree of dampness and the measure of the water-content was reduced correspondingly. In the early stages of the work a falling-off was noted in the compression tests of the concrete, which was finally traced to the sand. The sand satisfied the sieve test and it appeared to be of good quality and clean. It was tested with a solution of sodium hydroxide and was found to contain organic impurities injurious to concrete. Thenceforward samples of all sand used were subjected to the colorimetric test.

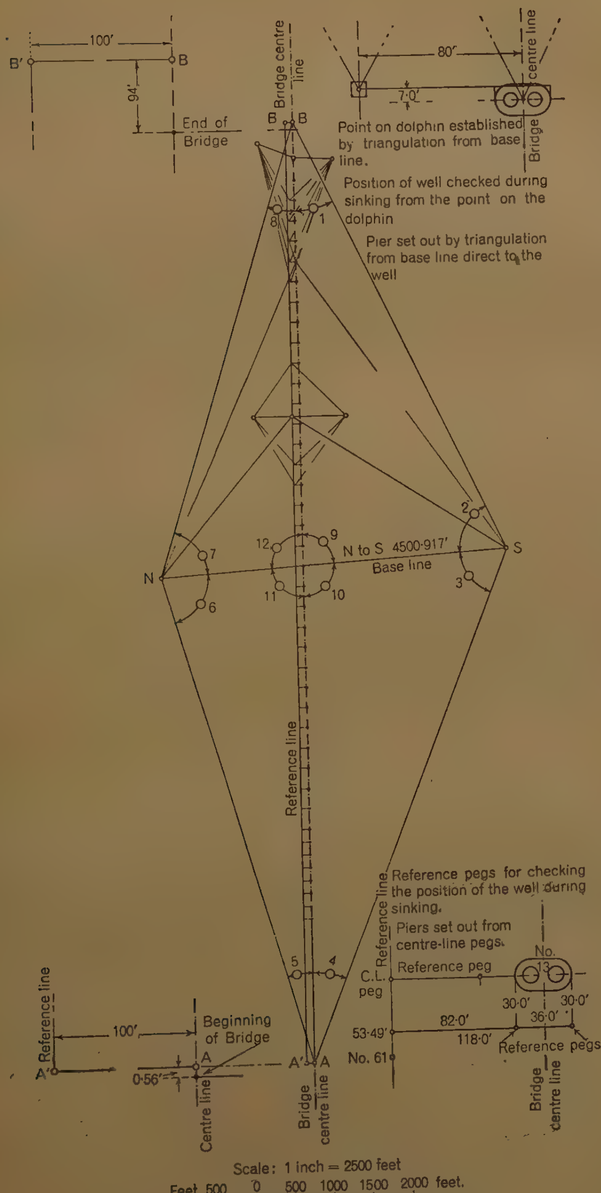
Setting-Out.

The location of the bridge was fixed by a beacon A on the right bank of the river and a beacon B on the left bank (*Figs. 26*). The distance between beacons A and B was computed to be :—

	Feet.
(*a*) By means of quadrilateral triangulation (Resident Engineer's staff)	12,157·610
(*b*) By means of simple triangulation (*Figs. 26*) (Resident Engineer's staff)	12,157·729
(*c*) By means of simple triangulation (Contractor's staff) .	12,157·768

The beginning of the bridge was fixed at a distance of 0·560 feet west of beacon A and the end at a distance of 94 feet west of beacon B. The distance between beacons A and B adopted from the above-mentioned triangulation was 12,157·768 feet, so that the length of the bridge was 12,064·328 feet. A reference line A′–B′ was set out 100 feet north of the line A–B and parallel with it for the purpose of setting out the wells and checking their positions during sinking

Figs. 26.



TRIANGULATION AND SETTING-OUT.

operations. Commencing at a point on this line at right angles from beacon A, pegs were set out 100 feet apart on the sandbanks. The distances were measured with an invar band and the pegs were given chainage values. Intermediate reference-pegs were established on this line for setting out and checking the positions of those wells which were to be sunk on the sandbanks. The positions of the wells sunk in water were set out by intersection from subsidiary base-lines, and piled dolphins were erected on or near the parallel line for checking the positions of the wells during sinking operations, from reference points established on the dolphins by intersection from the subsidiary base-lines. At low-river in 1933 the chainage on the parallel line A′–B′ was carried forward in the direction of beacon B on the sandbanks and a point was established opposite pier No. 30. The distance from beacon B to a point on the top of this pier on the line A–B was measured by an invar band along the spans which had been erected, and from that point a line was set out at right angles from the line A–B intersecting the parallel line A′–B′. The necessary adjustment was made and the total distance from beacon A to beacon B was thus arrived at by direct measurement; it was 12,158·004 feet as compared with 12,157·768 feet obtained by triangulation. The difference of 0·236 foot is equivalent to 1 in 51,500. The erection of the spans was carried on from both ends of the bridge. The bearing plates were set out by direct measurement from the respective ends of the bridge and the closing difference was found to be 0·031 foot, which is equivalent to $\frac{3}{8}$ inch.

Painting.

The specification provided that the steelwork, excepting the trestles and bracing of the viaduct, which would be partly under water, should be scraped perfectly free from rust, scale and dirt, well brushed with steel brushes and given one heavy coat of Griffiths Brothers' "Natural Ferrodor" paint, or similar paint of other approved make, at the contractor's works. The specification further provided that after erection on site the structural steelwork should be well cleaned by scraping and brushing, and painted with two more coats of "Natural Ferrodor" paint. Briggs's "Tenax" solution or other approved protection was specified for the Phœnix columns and bracing of the viaduct. The scraping, cleaning and brushing was carried out satisfactorily both at the contractor's works and subsequently on site, and although the steelwork had been freed from mill-scale as far as could reasonably be done, it could not be said to be perfectly free from it. It could be said, however, that its condition was such as satisfied general practice.

Griffiths Brothers' "Natural Ferrodor" paint was used, and the superstructure of the viaduct, the approach-spans and thirteen main spans were painted with two coats on site. At this stage of the painting the Author became very uneasy about the appearance of the paintwork, which seemed to be breaking down with abnormal rapidity. A dull black powder appeared in numerous patches all over the paintwork from 4 to 5 months after painting; 9 months after painting, rust stains were showing through the two-coat work and, in some places, the paint films were beginning to lift and scale off. The condition of the paintwork appeared to be so serious that all painting on site was stopped and the matter reported to the Consulting Engineers.

As a result, the paint manufacturers immediately sent their chief chemist to the site. He agreed that the scraping and cleaning and the application of the paint had been done in a satisfactory manner and that the rapidity of the breakdown of the paint was abnormal. He was puzzled by the black powder on the surface of the paint, which he could not account for until its composition was known. Samples of the black powder and films of the paint were collected and taken to London by the chemist, and were submitted to Dr. L. A. Jordan, Director of the Paint Research Station, for examination.

It was found that the paint was infected by air-borne fungus spores and bacteria of a particularly virulent type; the fungus was a mycelium and the bacteria were of a type associated with soil and decaying vegetable matter. The bacterial condition of the paintwork explained its rapid breakdown, and the early destruction of the paint film accounted for rust spots having appeared 9 months after painting. The feeding process of the bacteria brought about a great change in the composition of the organic residue of the black powder which, as revealed by the analysis, yielded only a small amount of oily matter in comparison with the original oil. In view of the importance of the matter and of the fact that similar paint had been used successfully in many parts of the world under similar severe climatic conditions to those which obtained at the site of the bridge, Dr. Jordan's report to the paint-manufacturers is given below in extenso, with his permission and that of Messrs. Griffiths Brothers, Ltd. :—

REPORT UPON EXAMINATION OF A BLACK POWDER SCRAPED FROM CERTAIN PAINT WORK.

The sample was received in a glass receptacle and it was examined before information was given as to its history.

It has been learned since that

(*a*) the black powder began to appear on Ferrodor paint 4 months after application in 2 coatwork, to a bridge in Tropical Africa.

(*b*) some concern has been expressed as to the state of the scale on the metal before painting, presumably with the thought that the paint film might have been affected thereby.

Chemical Examination.

An ultimate analysis was made and later compared with the ultimate analysis of an ordinary dried film of Ferrodor paint. The results of analysis were as follows :—

	Ash.	Carbon.	Hydrogen.	Oxygen.
Black Powder	31·4	38·2	9·4	21·0
Ordinary dried Ferrodor Paint .	67·4	9·8	2·3	20·5

As far as could be judged without making a most elaborate analysis, the ash in each case was practically identical in composition. It contained Fe_2O_3, Al, Mg, and traces of Sn or Sb and SiO_2 and corresponded to what would be expected of ignited micaceous iron ore. The iron oxide was in both cases non-magnetic.

It appeared, therefore, certain that the inorganic content of the black powder as to quality was what would be expected from Ferrodor paint. The quantity has no significance under the circumstances.

The organic part of the black powder was very different from the organic part of a Ferrodor paint film. A portion of the black powder was extracted with benzyl alcohol (which will dissolve oxidised oil) but only a small yield of oily matter was obtained indicating that the organic part, if it ever had been oil, was now very much changed.

The high carbon content of the ultimate analysis suggested at first sight bituminous material but complete insolubility of the black powder in bisulphide of carbon proved the absence of bituminous substance.

Microscopic Examination.

Microscopic examination besides confirming the appearance of micaceous iron ore revealed the presence of fungus spores, stray fibres of cellulosic material and possibly particles of sand.

A proper bacteriological examination revealed true bacteria as well as fungus spores, both types of which have been grown and counted. To make this examination, the material was soaked in 2 per cent. saline solution for 2 hours and aliquot portions transferred to gelatin plates for bacterial growth and to wort agar for fungus growth.

Counts indicated the presence of

10 million spores of mycelium per gram of powder, and
500,000 bacteria per gram of powder.

The fungus is definitely a mycelium, and whilst the bacteria has not been given a name it is definitely a type associated with soil and decaying vegetable matter.

It was subsequently ascertained that not only was the "Natural Ferrodor" paint applied on site infected but that the time-hardened shop-coat also became infected after delivery to the site. It was also discovered that the infection was not confined to the "Natural Ferrodor" paint but that the ordinary red lead used for painting the joints of the steelwork became infected. Further experimental laboratory work yielded evidence that the fungus component is responsible for the beginning of the attack, which opens up the surface for further bacterial development, and that the whole operation of fungus attack requires the presence of air and water. Anærobic conditions do not permit the survival of the micro-organisms. Fungus attack generally means saponification of the oil, which is followed by other types of decomposition through oxidation and similar processes; this type of decomposition soon renders the film more porous and water-attractive, and rapidly develops favourable conditions for a much more intensified form of attack. A drum of the paint supplied was sent from the site of the bridge to London for independent analysis; it was reported to be in good condition chemically and to be of such a composition that it should have excellent protective qualities.

Experimental laboratory tests were continued by the technical staff of the paint manufacturers and at the Paint Research Station, with films of paint and the black powder sent from the bridge, and it was reported that normal paints made respectively of red lead, white lead, red oxide and zinc oxide, with mediums of ordinary boiled oil and also of stand oil, had all been infected by the black powder. There was, therefore, evidence for the conclusion that any paint which would have been considered suitable for the painting of the bridge would have been attacked, and would have "broken down" under the abnormal conditions prevailing at the site. Attempts were made to produce with normal ingredients a paint

with a resistance to saponification so high as to confer immunity from attack, but all the attempts failed.

Concurrently with the above-mentioned experiments, numerous paints were made involving the inclusion of various fungicides in the formulas, but in doing so the following technical difficulties had to be provided for:—

(*a*) The fungicide must be effective vis-à-vis the particular spores and bacteria in the black powder.

(*b*) It must be capable of wide dispersion throughout the medium; in other words, it must be soluble in the medium.

(*c*) It must not react chemically with any other ingredient in the paint; the result of such reaction would destroy it as a fungicide.

(*d*) Its use must be compatible with the maintenance of high protective quality, reasonable ease of application and rate of drying in the finished product, and should not involve undue risk of poisoning to the maker and the user.

The conclusion arrived at, as the result of the experiments made, was that only by the use of a fungicide could a paint be produced which would be effective under the abnormal conditions prevailing at the site of the bridge, and which would resist attack by fungus and/or bacteria at present on the bridge and by any that may be air-borne on to it later.

The paint-manufacturers succeeded, 2 months after Dr. Jordan's report, in producing a "Natural Ferrodor" paint with the highest saponification resistance practicable and containing a fungicide of copper compound, and it remained uninfected after 1 month's exposure. The paint remained uninfected after exposure for another month and a consignment was thereupon dispatched to the bridge for application. On raising the question as to the poisonous nature of the paint in respect of persons using it, the paint manufacturers stated that the new preparation is much less poisonous than many anti-fouling paints and that it is only likely to cause trouble if taken internally. At the same time they gave a warning against natives mixing the paint with their fingers, as they often do, and afterwards using their fingers for eating.

A close examination of the spans which had not been painted on site showed that comparatively little paint had been removed during the journey from England and during erection. The steel-work was therefore fairly well protected immediately following erection, but thereafter the degree of protection gradually diminished

by the loosening of the scale underneath the shop-coat. Owing to the protection which the shop-coat afforded the mill-scale (which it had been impossible to remove before painting) and the hardness which the paint had acquired, the scraping and cleaning on site had very little effect beyond the removal of loosely-adhering mill-scale, rust and paint. The major portion of the mill-scale and paint was still firmly adherent, but during the subsequent weathering period it was possible to remove it easily with wire brushes.

It was not suggested that the complete removal of the mill-scale before painting would have prevented the infection, and this was demonstrated by experimental painting on some parts of the steelwork from which the mill-scale and the shop-coat had been entirely removed; the paint on these parts became infected in due course. The paint-manufacturers, however, expressed the opinion that when the cleaning on site of the steelwork was finished it was certainly clean in the sense that nothing more could be removed, but that it was by no means a suitable surface on which to apply paint, and they strongly recommended that descaling should be allowed to take place by the natural process of weathering before the final scraping, brushing and painting of the bridge with the new anti-fungus preparation. This condition is no doubt ideal and may be considered to be sound practice; although it will be possible to give effect to the recommendations in the case of this bridge, the conditions under which bridges in general are constructed and painted do not always permit the necessary time to elapse for weathering on site before the painting is done.

The destruction of the original paint by micro-organisms was regarded as being of such importance that the Directors of the Paint Research Station and of the Chemical Research Laboratory have joined in an application to the Department of Scientific and Industrial Research for a grant to finance general research on the subject.

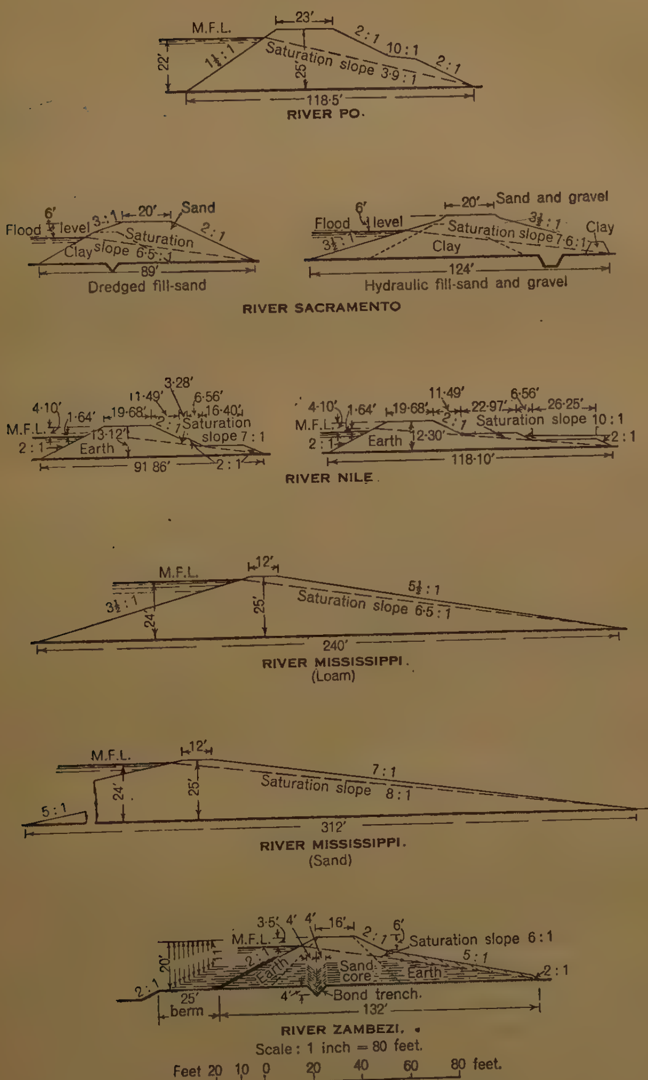
The Connecting Railways.

South Connecting Railway.—Two routes were studied for the new railway to link up the bridge with the Trans-Zambesia Railway. The major feature which had to be taken into consideration was that the country over which the railway would pass was subject to inundation for a distance of 20 miles inland when the river Zambezi was in high flood, and that the railway would have to be constructed across the area subject to inundation for half its length or more. The route finally decided upon left the Trans-Zambesia Railway near its terminus at Murraça and followed along the margin of the

river Zambezi to Sena at a minimum distance of 2,000 feet from the river. The line is 25·29 miles long, 20 miles of it passing over the area subject to inundation. Although the earthworks were heavier than they would have been for the alternative route, the additional cost was more than compensated by the line being 3 miles shorter and by a reduced expenditure on bridges and culverts above flood-level.

For obvious reasons it was out of the question to provide any openings in the embankment which would admit the flood-water of the river Zambezi, and as the minor rivers and streams in the area subject to inundation discharge into the river Little Zangwe, there was no necessity to do so. The new embankment was therefore required not only to exercise the functions of a railway embankment but also to act as a levee for the river Zambezi, and consequently the customary design for a railway embankment would not meet the case. It was necessary to establish a slope of saturation within the embankment at maximum-flood level and to provide sufficient cover on the land side to resist the breaking through of flood-water by percolation and the sliding of the embankment along the line of saturation. The maximum head of water to be provided for against the embankment was 15 feet. *Figs. 27* shows standard cross sections of levees on some well-known rivers. A saturation-slope of 7 to 1, which, if measured from maximum-flood level is equivalent to 8 to 1, satisfies (excepting in extreme cases), the conditions obtaining on the river Nile, but a levee on the river Mississippi failed with a saturation-slope of 7·58 to 1, and on the river Po a saturation-slope of 4 to 1 was not always effective. Comparisons of the saturation-slopes at different sites, important as they are, were not a sufficient guide in deciding upon the cross section of the embankment for the South Connecting railway without taking into consideration the method of constructing the levees, whether by manual labour or mechanical means, and without considering the nature of the material available. The material available was favourable for the construction of an embankment of high density and low degree of permeability and for the most part consisted of very fine sand and fine sandy loam. There were some short stretches where the material available was a dark colloidal alluvium and at these places the central part of the embankment was required to be constructed with fine sand. The specification called for all growth and vegetation to be cleared and the area to be occupied by the embankment to be ploughed or dug over and any soft ground removed. The embankment was required to be constructed in horizontal layers 12 inches in depth extending throughout its entire width, and all clods of earth were required to be broken to the size of a man's fist. The usual method of con-

Figs. 27.



STANDARD CROSS SECTIONS OF LEVEES.

structing railway embankments in Central Africa is by natives who carry the material in baskets or pans, and during this process it is well tramped down.

In view of the favourable material and method of construction prescribed it was decided that a saturation-slope of 6 to 1 would satisfy the conditions. A slope of 2 to 1 on both sides of the embankment gives the required saturation-slope with ample cover on the land side, for embankments up to 11 feet in height. For embankments above that height the slope or banquette of 5 to 1 also satisfies the saturation-slope and varies in length directly with the height of the embankment. On the river side, the embankment is protected against erosion to high-flood level by stone pitching 12 inches thick consisting of basalt stones weighing from 65 to 165 lbs. each. A V-shaped trench 8 feet wide at the top and 4 feet deep is excavated in the ground on the site of the embankment and parallel with it to ensure a bond between the ground and the embankment and to prevent seepage along the base of the latter. The material for the construction of the embankment was obtained from borrow-pits on the river side of the embankment. A berm of 25 feet was left between the toe of the embankment and the borrow-pit, and the borrow-pits were separated by a berm 60 feet wide.

The highest head of water against the embankment up to the present time has been 11 feet and at that particular place the embankment was a little over a year old. A normal amount of settlement took place, but the embankment suffered no damage, and there were no signs of escape of water by percolation through the embankment. At several points a small amount of "piping" took place through the sub-soil beneath the embankment and water could be observed oozing up through the ground from 6 to 40 feet from the toe of the embankment on the land side, but it was not of a serious nature.

The existing Trans-Zambesia Railway, which was connected up with the new line at Murraça and which was a surface line, was subject to flooding by the river Zambezi at various points over a distance of 8½ miles south of Murraça, and it was resolved to raise the formation level to a height corresponding to that of the south connecting railway; the whole of the railway system serving Nyasaland would then be free from flooding. At the same time it was decided to carry out a re-alignment of the Trans-Zambesia railway which would effect a saving in distance. The length of the section which was re-aligned is 8·61 miles which, added to the 25·29 miles of new line, made the length of new construction 33·90 miles, 28·37 miles of which is on continuous embankment. The volume of earthworks was 2,844,898 cubic yards, 2,835,010 cubic yards of which was embankment and 9,888 cubic yards in cutting.

North Connecting Railway.—The north connecting railway links up the bridge with the Nyasaland Railways at Baue and is 2·91 miles

long. The line is not subject to flooding by the river Zambezi and the construction of the railway presented no unusual features.

Costs.

The cost of the respective parts of the bridge, calculated at the contract prices, is given below. The cost per square foot is calculated on the overall width of the spans. The cost of the track is not included.

	Per square foot.			Per linear foot.		
	£	s.	d.	£	s.	d.
Main wells, piers and spans	5	17	10	113	7	10
Secondary wells, piers and spans	5	3	4	94	11	0
East approach	4	0	2	24	7	6
West ,, (viaduct)	5	3	2	25	7	8

Partial costs, included in the above, were as follows:—

	£	s.	d.
Well-sinking:			
Main wells. By open dredging (3,499 feet)	32	11	1
Secondary wells. By open dredging (548 feet)	12	17	5
Main wells. Under compressed air (91 feet)	118	10	9
Secondary wells. Under compressed air (142 feet)	90	10	8
Footway, trolley refuges and staircases (per linear foot of the bridge)	4	16	8

The ratio of the cost of the superstructure to the cost of the foundations and supports was 0·92 for the main spans and 0·84 for the secondary spans.

The cost of the bridge under the contract was £1,162,000 and the expenditure on the connecting railways was approximately £400,000.

Conclusion.

The Consulting Engineers were Messrs. Livesey and Henderson and Messrs. Rendel, Palmer and Tritton. They acted independently during the preliminary investigations, but for the purposes of the design and construction of the bridge they acted jointly and were represented by the Author as Resident Engineer, with Mr. H. E. Whitehouse, M.C., M.A., Assoc. M. Inst. C.E., as Chief Assistant. The Cleveland Bridge and Engineering Company, Limited, were the contractors for the whole of the works, which were carried out under the direction of Mr. John Coupland, M. Inst. C.E., with Mr. G. E. Howorth, M.C., B.Sc., M. Inst. C.E., as Agent at the site of the bridge.

The Author is indebted to the Joint Consulting Engineers for permission to submit the Paper; to Mr. F. Dixey, O.B.E., D.Sc.,

F.G.S., Director of the Geological Survey of Nyasaland, for reviewing the section on the geology of the district; to the American Society of Civil Engineers for information concerning the levees on the river Mississippi and other levees; to the Under-Secretary of State to the Egyptian Government for information in connection with levees on the river Nile; and to Mr. W. G. Fairweather, B.Sc., Assoc. M. Inst. C.E., Director of Surveys to the Northern Rhodesian Government, for information concerning the upper reaches of the river Zambezi. The Author wishes to express his gratitude to the late Sir Brodie H. Henderson, K.C.M.G., C.B., Past-President Inst. C.E., for the confidence he placed in him and for his kindly advice on any difficulty which arose during the construction of the bridge, and to acknowledge the assistance he received from Mr. A. J. W. Learmouth, B.E., Assoc. M. Inst. C.E., and other members of the Author's staff, in the preparation of matter for this Paper. The Author also wishes to pay a tribute to the cordial relations which existed throughout the construction of the bridge with the Portuguese officials of all grades, both of the Colony of Moçambique and the Moçambique Company, and to the assistance which the local and higher authorities were always ready to give in dealing with any difficulties which arose.

The contract was signed on 17 October, 1930, and the works were in a sufficiently advanced state to permit of running a full train service on 1 March, 1935, or nearly 7 weeks before the expiry of the contract time of 4½ years. The bridge was finally completed on 17 September, 1935, with the exception of the painting.

The Paper is accompanied by thirty-nine sheets of drawings and three wall-diagrams, from some of which Plates 1 and 2 and the Figures in the text have been prepared, and by three photographs.

Paper No. 5055.

"The Construction of the Lower Zambezi Bridge." †

By GEORGE ERIC HOWORTH, M.C., B.Sc., M. Inst. C.E.

TABLE OF CONTENTS.

INTRODUCTION.

On 17 October, 1930, contracts were entered into by the Cleveland Bridge and Engineering Company for the construction of the Lower Zambezi bridge and the railway to connect it with the existing system of the Trans-Zambesia Railway. The contracts were to run concurrently, the completion date in both cases being 16 April, 1935. The purpose of this Paper is to describe the general arrangements made for carrying out the contracts and the methods of working adopted, and to record the progress of the works from the contractors' point of view.

PRELIMINARY ARRANGEMENTS AT THE SITE.

On 6 December, 1930, an advance party of the contractors' staff arrived at the bridge-site. A visit to the site had been made before

† Correspondence on this Paper can be accepted until the 15th May, 1937.
—SEC. INST. C.E.

the preparation of the tender, but owing to the very short interval between receipt of the invitation to tender and the originally intended depositing date for tenders, it had only been possible to spend a very few days actually at the site, and nearly all conclusions arrived at were in need of confirmation and elaboration. The main points to be dealt with urgently were :—

(1) Sources of supply of personnel and labour, and arrangements for working conditions, feeding, and housing.
(2) Location of contractors' camp, yards and workshops, and the putting in hand of water-supply, sanitary and health arrangements.
(3) Layout of communications, including water-transport, landing-places, and permanent and temporary railway-tracks.
(4) Confirmation of sources of stone-supply as regarded quantity, variation in quality, distribution and accessibility to site, and quarrying methods to be adopted.

Personnel.

The contractors' home-engaged works staff which had to be provided for consisted of engineers, foremen, office and stores staff, and a medical officer. For the supply of subordinate staff the local European population was relied upon, and it was evident from the beginning that there would be no shortage as regarded numbers applying for employment. The men were, however, almost entirely without skill or previous experience on large works carried out under proper control, and had to be taught. It was decided to employ Europeans as drivers of all well-sinking cranes, and upwards of thirty men were required for this service alone. Compressed-air overseers, of whom there was always a serious shortage, had to be taught from the beginning. The men were drawn from all the surrounding territories, from Tanganyika and the Rhodesias to the Cape Province. In all, about one hundred and sixty local non-natives were employed at one time, including Mauritian and Goanese fitters. Many of them stayed for a very short time only. Others worked right through the job. No Indian labour was at any time employed on the works, and such work as pneumatic riveting, locomotive- and stationary-engine driving, rough fitting and carpentry was all done by African natives.

Camp Arrangements.

As soon as the general layout of the works had been decided upon, a suitable site near the left-bank bridge-head was selected for the

European camp, stores and offices.[1] In planning the camp great attention was paid to health matters, more especially in connexion with water-supply, sanitation and the prevention of malaria. All water had to be taken from the river and required a very considerable amount of treatment before it could be called potable, but eventually an excellent water was obtained. With regard to sanitation, wherever possible the fuming-pit system was adopted for native latrines and a chemical system for Europeans. Particular attention was paid to cleanliness and protection from fly infection. Slaughtering was done in a fly-proof abattoir. The resulting freedom of the site from flies was very marked. Anti-malarial measures were undertaken by the contractors. The advice of the Ross Institute was obtained, and Mr. C. R. Harrison visited the site and made recommendations which formed the basis of the system adopted. The underlying principle advocated by the Ross Institute in this case was simplicity itself. It was almost entirely a question of drainage and inspection for water, the underlying principle being "no water—no mosquitoes." In practice, however, such measures call for constant vigilance, the whole-hearted co-operation of all concerned, and an elaborate system of control with a view to showing up any weak points in the defences before a serious influx of mosquitoes occurs. There can be no question as to the justification of expenditure on such measures. The extra efficiency of a staff free from malaria is worth far more than the cost of the protection. Among the contractors' home-staff only four cases occurred, although previously the district had had a deservedly bad reputation for malaria. In three of these cases there is little doubt that infection was communicated while the individuals concerned were living outside the protected area.

Everything reasonably possible was done for the comfort of the staff; quarters were comfortably furnished, ice-making and cold-storage plant, electric light, fans, etc., were provided, and later, as opportunities arose, recreation facilities were added in the form of a clubroom and billiard-table, tennis-court, squash-court and a 9-hole golf-course. Although working in a very trying climate, the contractors' home staff, at any rate, kept remarkably fit throughout the period of the work, in spite of, or perhaps to some extent because of, the very long working-hours which most of them put in. The contract called for the employment of a doctor with Portuguese diplomas, and the contractors were fortunate in obtaining the services of Dr. José Oliva. All medical and health matters received his

---

1 *Fig. 3*, p. 331.—SEC. INST. C.E.

personal attention, and the results bear testimony to the excellence of his work.

Labour.

Unskilled native African labour was readily obtainable from all surrounding territories. Very little trained, skilled, or semi-skilled native labour was available, and it was found to be much more satisfactory in all cases where skill was required to get absolutely raw natives and teach them from the beginning. The African native is very adaptable and learns quickly. So long as he is kept on one job and one job only he is capable of turning out really good work if properly taught. Natives from very many different tribes were employed on the works, the main sources of supply being the Zambezi Basin, Nyasaland, the district of Quelimane, and the territory of the Moçambique Company. Natives were housed in large carefully-sited compounds consisting of well-built reed-thatched huts of the general pattern to which they are accustomed, but better built and with considerably more cubic space per head, five single men being accommodated in a circular hut 13 feet in diameter and 7 feet high to the eaves with a high pointed thatched roof. Adequate fuming-pit sanitary arrangements were provided, also stone and corrugated-iron ration stores, a good water-supply, and an efficient native hospital with operating theatre and all necessary equipment in conformity with modern standards. A small force of native police was employed by the Company direct, but all cases requiring trial and punishment were handed over after preliminary inquiry to the nearest authorities.

Minimum scales of pay and rationing were fixed by law in both of the territories in which the contracts were carried out, but difficulties arose from the fact that the minimum scale of pay in one territory was greater than that in the other. The same law, common to all Portuguese Colonies, governed this matter in both territories, but the monthly scale of pay was fixed as a percentage of the annual hut-tax payable, and since the hut-tax differed in the two territories, the minimum rate of pay was also different. As the railway construction was entirely in the Moçambique Company's territory the conditions there were definite. The bridge construction, on the other hand, although carried out under a concession from the Portuguese Government direct, involved the employment of labour in both territories, and difficulties were of frequent occurrence. These matters, and the somewhat similar question of customs-control, required early attention and gave a lot of trouble throughout the work.

Communications and Construction of South Connecting Railway.

All material and plant for the works were brought from overseas and landed in Africa at Beira.[1] The bulk of the earlier plant was required on the left bank, where there was a suitable landing-place just above the bridge site. On the right bank there was no reliable landing-place within 5 miles of the site. This consideration, together with the fact that most of the difficult work was likely to be in connection with compressed-air sinking on the wells near the left bank, had a considerable influence on the decision to locate the contractors' main yard, offices and camp on the left bank. The loads transported up river were very limited on account of the difficult navigation, which necessitated the use of small shallow-draught craft. The amount of handling to which materials were subjected in loading and offloading was excessive and the costs correspondingly high. The railway company themselves had made a commencement with the construction of a part of the permanent embankment near Murraça by petty contract, but this arrangement terminated on the letting of the main contracts.

The new railway alignment runs parallel with the river, and the embankment, which for 25 miles continuously has no spill-openings, traverses an area which has always been subject to deep flooding from the Zambezi. The flood-area extends for about 20 miles back from the normal river-bank so that the potential volume of water which might, during a flood, overflow behind a partially-completed embankment was practically unlimited; it was therefore regarded as being of the greatest importance that the work on the Inharuca—Murraça section, once restarted, should be completed before the next flood-season. Any opening left unfinished must be a source of grave danger, and the smaller the opening the greater the risk. A very early start was therefore desirable. The work was accordingly recommenced by the contractors immediately on their arrival at site in December, 1930, but as this involved carrying on through the 1930–31 flood-season it was recommenced from the Inharuca or upstream end, where the ground-level for the first few miles was above anything except a very high flood-level; and the sites of the borrow-pits were generally dry. The whole of the earthwork of the embankment between Murraça and Inharuca was actually completed at the end of January, 1932, and the track linked up and communication established a week or so later. Up to that date approximately 1½ million cubic yards of earthwork

[1] *Fig. 1*, p. 327.—Sec. Inst. C.E.

had been carried out by head-loads from borrow-pits and 23,000 cubic yards of stone pitching placed in position.

From that time onwards all permanent materials, except such as were definitely required for erection on the left bank, were hauled up the permanent railway by the contractors' locomotives and handled at Sena, where a second stockyard and cement store were provided.[1] Stone for concrete aggregate from a quarry at Inharuca was also transported over the permanent track between Inharuca and Sena. The Sena stockyard was situated near the right bank abutment. From this point a temporary track of serviceable 41¼-lb. rails capable of carrying the 6-coupled construction locomotives was laid in the dry river-bed parallel with and downstream of the bridge centre-line to the high sandbank forming the edge of the normal main channel downstream of the site of pier No. 15. This involved the crossing of several minor channels and backwaters on temporary sand embankments which were, of course, washed away during each flood-season, but in order to reopen some communication as soon as possible after such washaways a light timber-pile gantry was put across the most important of these channels. The track served as a link in the cross-river communication at the bridge-site in addition to its main function of feeding materials to construction work carried out in the dry on that part of the bridge. Several other service tracks of from 1 to 5 miles in length were laid at various times to connect up outlying quarry sites with the permanent railway system.

After the linking-up of the bridge site with the existing railway system at Murraça, all work on railway construction was suspended for 15 months on account of the heavy exchange losses which were being sustained by the contractors consequent on the Moçambique Company's adherence to the gold standard. The Government Territory, in which the bridge itself was being constructed, followed Portugal and Great Britain and exchange for this portion of the works was not affected. Work on the railway was not resumed until April, 1933, when the Moçambique Company followed South Africa off gold.

Stone-Supply.

The stone requirements for the purpose of the contracts were as follows :—

Pitching-stone for railway embankments . .	41,000 cubic yards.
Pitching-stone for protection of bridge piers from scour	53,000 ,,
Aggregate for concrete in well-steining, etc. .	95,000 ,,

[1] *Fig. 3*, p. 331.—Sec. Inst. C.E.

For concrete aggregate for well-steining, basalt was specified on account of its high specific gravity. For pitching-stone of either class, basalt or other approved stone was specified; the minimum weight of any stone was to be 68 lbs.

At the bridge site and for a long distance back from the river on the right bank the main geological formation consists of thick beds of soft red sandstone alternating with rhyolites, lava, and volcanic ashes, and generally covered with alluvial material which forms a low-lying plain extending over a large area. On the left bank the same sandstone formation is exposed at the surface in broken rising ground to a line several miles back from the river, where it is interrupted by a considerable fault running roughly parallel with the river. A very noticeable feature of the country, particularly on the right bank, is the large number of scattered conical hills of heights up to 400 feet standing up out of the alluvial plain. In the distance they present more the appearance of deserted mining dumps than the natural formations which they really are. Their exposed slopes consist of denuded sandstone and debris, but in each of the hills there are more or less vertical cores or pipes of basalt varying in diameter from a few feet only up to about 140 feet, to the non-erodible qualities of which the hills owe their existence.[1] The pipes are actually the vents through which molten basalt has once found its way to the surface, although at much higher levels than the surface existing to-day. These pipes offered the only practicable source of supply of stone for all purposes. On the left bank only two occurrences of pipe basalt were found in the vicinity of the works. One, known for works purposes as No. 1 Hill, was close to the north connecting railway and only a few hundred yards back from the left bank bridge-head; the other, No. 2 Hill, was close to the river bank about 2 miles upstream. On the right bank half a dozen or more conical hills containing pipes occurred within a distance of 6 or 7 miles from the bridge, the two largest and most promising being Sena Hill, about 1 mile inland, and Inharuca Hill, about 5 miles downstream.

It was evident that quarrying would present unusual features. In the first place no very large quantity of stone could exist in any one pipe or group of pipes, and even for comparatively small quantities overburdens would be heavy and rates of output restricted. It would obviously be necessary to work a number of small quarries together. Again, in consequence of the unusual formation showing no exposed vertical faces, it was only possible before the quarries

[1] The MS. contains further information on the geology of the "pipes," and may be seen in the Institution Library.—Sec. Inst. C.E.

were opened up to make a very superficial examination of the stone in the very small outcrops at the tops of the hills; on account of the severe weathering in that situation it was difficult to estimate quality, and impossible to estimate the proportion of stone of sizes suitable for pitching to total production. It was clear that the stone from all pipes contained many cleavage planes, and from some it was very highly shattered. The only useful guide to proportions of pitching-stone available appeared to be the relative condition and sizes of the basalt debris which had broken away from the pipes and was lying on the slopes of the hills mixed with the sandstone debris. The basalt debris itself at Inharuca quarry provided a useful supplementary supply of stone.

As work was to be started simultaneously on both banks of the river and the river-bed and water-conditions were such as to make continuous cross-river communication by water extremely unreliable, it was evident that quarries must be opened on both banks. On the left bank No. 1 Hill was opened up for concreting-stone only. On the right bank both Sena and Inharuca were opened up, and it was hoped that a sufficient proportion of large stones would be available from these two hills to provide the pitching-stone definitely required for the railway embankment, and at least for that part of the bridge pitching which would be placed in the dry river-bed at times of low river. No. 2 Hill was kept in reserve as a source of supply for pitching-stone to be transported and placed around piers in deep water by floating craft; if necessary, in the event of the right-bank sources of supply giving out, it was available for the pitching around such of the other piers as remained to be done after communication over the completed bridge had been established. Actually, the supplies of pitching-stone from the right bank fell far short of requirements, and, in addition, quarrying conditions at No. 2 Hill itself became so difficult that it eventually became necessary to obtain supplementary supplies.

From the working point of view, except in No. 2 Hill, the stone barred out fairly freely once an open face was obtained.. Although heavy pneumatic drilling plant had been provided, experimental operations showed it to be practically impossible to get deep holes through the layers of excessively hard tough stone alternating with shattered and brittle material, and it was never used, the only shot-firing taking place being for the removal of sandstone overburden and for breaking down big boulders. The output was never sufficiently concentrated or certain at any point to justify the installation of inclines or heavy plant for getting the stone down the hills, and the stone was simply rolled down the hills in stages and collected at the bottom. In spite of the apparent danger of this procedure

the accident rate was low. With the cheap labour available, normal production-costs were reasonably low and it did not appear at any time that they could be improved by the application of more civilized methods. Crushing machinery was installed at convenient places for feeding near the foot of each hill, and the crushed stone was taken by rail direct from the crusher-hoppers to the dumps at the mixers. The large stone of pitching size was rolled into heaps by hand labour and later rolled by hand into flat rubble-skips, which were picked up by 5-ton locomotive-cranes and loaded into 10-cubic-yard steel tip-wagons for transport to the site. The quantities which were eventually taken from the several quarries were as follows :—

	Pitching stone : cubic yards.	Concrete aggregate : cubic yards.	Approximate overburden and ridd : cubic yards.
No. 1 Hill	—	26,662	25,000
No. 2 Hill	27,000	—	40,000
Sena Hill	8,725	26,905	20,000
Inharuca Hill	48,403	40,910	20,000
Other sources	11,000	—	9,000

Survey for Length.

As it was decided to commence construction at both ends of the bridge an accurate survey of the site for centre-line length was put in hand, the accuracy of all setting-out work being the contractors' responsibility. An excellent site for a base-line about 5,000 feet long up- and downstream existed on a high sandbank well above ordinary high-river level at about the middle of the main spans. A base-line was set out and carefully levelled on this bank. It was intersected by the bridge centre-line nearly at right angles near its own mid-point, and gave very good angles over the mile of low river waterway between it and the starting point of the bridge on the left bank. The two ends of the base-line were marked with heavy permanent marks and instrument platforms were constructed above them. The base-line was carefully measured by invar tape jointly with the Resident Engineer's staff and the length agreed before any further survey work was done. The five marks constituting the two ends of the base-line, the two ends of the bridge centre-line, and the intersection of the two lines, defined a well-shaped quadrilateral and its two diagonals. The reading of each angle was repeated six times, corrections were made, and the positions of all points computed and reported to the Resident Engineer. After checking and comparison with a previous survey carried out

by his staff on other lines, these positions were accepted and officially adopted as fixed points to which all setting-out work would be referred.

General Working Methods and Plant-Provisions.

From the data available it appeared that river-levels could be predicted with considerable certainty between April and the end of November. In April, after the flood-season, the level would be about 10 feet above low-river level and the river would be contained between the spill-banks, leaving a waterway at the site of the bridge approximately 1 mile wide. If the position of the main channel did not alter during the period of construction of the bridge, the sites of about half the main wells at the Sena end of the bridge, except for a few wells which were sited in comparatively small backwater channels, would be dry in April and would remain dry until just after low river in November. The 10-foot fall from April to November was gradual and practically uniform. During the later part of November or early December it could be predicted with confidence that the river would rise suddenly from low-river level back to about its April level. Between December and April the river-level could not be foretold within 15 feet; flood-peaks might occur at indefinite dates, on two or three occasions, and at any time during a flood-season the river might rise up to a maximum of 25 feet above dead low river. The best that could be done by way of warning was to get daily telegraphic water-level records from near Tete, about 160 miles upstream, which allowed of a fairly accurate prediction 48 hours in advance. During the flood-season movements of 5 feet or more in 24 hours could be anticipated.

It was estimated that it would be possible to carry out the sinking of about sixteen of the main wells in the dry from sandbanks and the exposed river-bed at low river. The sinking of any of the remaining wells might call for subaqueous work in water subject to considerable variation in depth; for the latter operation it was decided that floating well-sinking sets should be used.

It was considered advisable to make the plant employed on the sinking-sets as far as possible interchangeable with that used for sinking in the dry, as the proportion of wet to dry sinking would vary. The type of sinking-set decided upon consisted essentially of a high steel working-platform or stage mounted on a pair of steel barges, coupled together in such a way as to surround the well being sunk. The staging was designed to carry two 8-ton steam derrick-cranes at a height sufficient to command all work on the well-sinking and subsequent construction of the piers, and in addition to provide a structure from which to suspend the well-curbs during the operation

of lowering them into position on the bed of the river. This latter operation was to be carried out by jacking. Provision was made at the deck-level of the barges for a temporary platform on which the assembling and riveting of the well-curbs would be carried out. The sinking-sets were provided with concrete-gauging and mixing plant, air-compressors for pneumatic sinking and riveting, hydraulic pumps, electric-lighting sets, etc., together with the boilers necessary for the supply of steam for all purposes. Four sinking-sets were supplied. For sinking in the dry additional 10-ton derrick-cranes to work from ground-level, and additional concrete-mixing plant, were provided, but it was intended that this provision should be supplemented by transferring plant from the floating sinking-sets as and when required.

It was known from borings that rock would be encountered, making compressed-air sinking necessary in at least five wells and possibly in very many more. Six air-locks were provided, together with the necessary shafts, and six domes of a special type called for in the contract. The wells were of the twin-shaft type, and this provision made it possible to use compressed air on any three wells simultaneously, each pair of locks being served by independent compressors. Two independent medical locks were supplied.

For the steelwork-erection of the main spans it was decided to use travelling goliath-cranes throughout. The goliath-cranes were to run on temporary steel staging, which would also carry the camber-jacks and support the permanent steelwork during erection. The temporary steelwork was specially designed for the work, and was carried either on groups of 12-inch by 12-inch timber piling or on cribwork, according to circumstances. Six spans of staging and two goliath-cranes were provided. This allowed either for erection from two points simultaneously or for more concentrated work from one point.

Projected Sinking Progress.

The plant-provisions outlined above were estimated on the rough assumption that two working-seasons would be available for well-sinking, the working-season lasting approximately from May to November. The capacities of plant-units were governed by the diameter of dredging holes and the displacement of the wells. The grab selected for the normal work, namely Stothert and Pitt's "Bacon" patent single-chain clam-shell type with hemispherical buckets with a capacity of 30 cubic feet flush and approximately 38 cubic feet heaped, was the largest convenient size for the diameter of dredging hole, and it was estimated that in normal sinking conditions through sand it could be relied upon to dredge the equivalent

of 5 feet in height of well per working shift. The routine working to be aimed at was:

one shift concreting,
one shift sinking,
one shift striking and fixing shutters.

The height of concrete lift was fixed accordingly at 5 feet and the capacities of concrete-mixers and cranes arranged to suit. From this it followed that, working two normal day-shifts and one night-shift in each 48 hours, the basic rate of progress per sinking set should be 2 feet 6 inches per day. If that rate could be maintained with the five sets provided the sinking and pier-construction would be carried out in two working-seasons of about 33 normal working-weeks each.

The above estimate was used, in the absence of any local experience of well-sinking conditions, as a very general approximation. Extra time would be required for founding and plugging, and for plant-movements and applications of, and working under, compressed air. On the other hand, much of the pier-construction could probably be carried out during the flood-season, and only one double shift in each 48 hours had been counted in, leaving an ample margin for adjustments of working-time. It was, moreover, very probable that the plant available could be spread out over more wells when sinking in the dry. It was hoped that one or two wells would be got down at the end of the first season, which would be occupied mainly in preparatory work, so that more accurate information would be available before the commencement of the first sinking-season proper.

Commencement of Plant-Deliveries and Construction of Barges and Steamers.

Deliveries of plant at the site commenced early in 1931, and in April, when sufficient material was available, a commencement was made on the building of the floating craft. As no pneumatic plant was available at this time the whole of the riveting was done by hand; this necessitated employing about four hundred natives and teaching them the work, as only a small proportion had done any riveting previously and then only on cold rivets of very small diameter. Upwards of 1¼ million ⅝-inch-diameter hot rivets were put in quite satisfactorily in this way. The first two of these barges were in the water by the first week in August, when a commencement was made with the superstructure of the first sinking-set, and all four sinking-sets were ready for work, with their complete equip-

ment of steamers and towing barges, before the commencement of the 1932 working-season.

Constructional Work.

Pier-Construction on Left Approach.

During June, 1931, work was commenced on the excavation of foundations, followed by the concreting of the left-bank abutment and the piers of the approach-spans on the left bank. These foundations were all in the dry in sandstone rock.

Well-Sinking.

Steelwork for the first two well-curbs was delivered at the site in July, 1931, and during August a commencement was made with their erection at the sites of wells Nos. 1 and 2, which were dry at that time. The curbs were erected approximately in their correct positions. Before sinking was commenced on any well, three reference-pegs were put in, one on the major axis and about 80 feet upstream, and two about 80 feet east or west from the major axis on lines set off square from the major axis at its upstream and downstream ends. These reference-pegs were checked periodically, and all routine checking of well-positions during sinking was referred to them. After founding and plugging of the wells, pier-positions were set out from the main survey-points direct on to the finished well-concrete. Before filling a well-curb with concrete it was sunk by hand excavation as far as standing-water level in the ground would permit, or until the top of the outer skin-plating was at ground-level. Great care was taken at this stage and during the first 20 or 30 feet of sinking proper to ensure accuracy of position and plumbing, and the position and cross levelling were checked by the overseer at practically every movement of the well. There was no need to fill in the steining concrete to obtain sinking weight at this stage, as the cutting-edges could be cleared by hand. The curb was certainly more manageable without it.

After a well has been sunk to a depth exceeding about one-and-a-half times its axis-diameter, it is, in the Author's opinion, generally a mistake to try to correct position in the plane of that axis, and attention should be concentrated on keeping the well accurately plumb. There are, however, occasions when a well tends to go out of plumb continually in one direction owing to more compact material being met with on one side more than on the other, and in such cases a slight over-correction in plumbing will both overcome this tendency and at the same time tend to check the accompanying side drift of the cutting edge. If any twist or screwing motion develops or any tendency to throw out of plumb which cannot be reconciled with

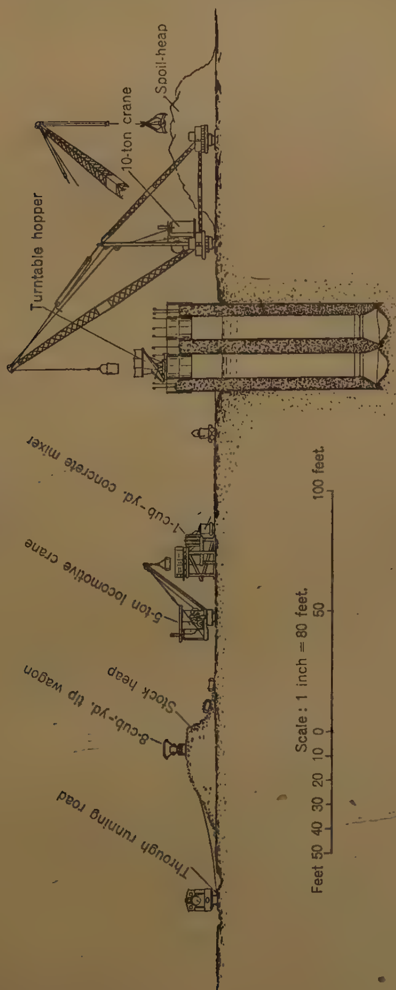
the quality of the material through which the well has been sunk or the ground at the cutting-edge (as shown by the excavated material), an obstruction should be suspected, and a diver's examination of the working chambers and of as much as possible of the cutting-edge is advisable. During the early stages the well should be kept light and the centre of gravity as low as possible. After the depth of penetration exceeds the length of the major axis by about 25 per cent., and provided that the well has been kept plumb to this stage and the ground to this depth gives solid side support, the more sinking weight the better.

When the top of the skin-plating had been sunk flush with the ground the curbs were filled with concrete which was then carried up about 8 feet above ground-level inside steel shuttering. Each ring of shuttering was 5 feet deep, and was made up of six segments, two straight and four curved in plan, bolted together at the vertical joints and to the next ring below, or to the well-curb in the case of the first ring, all around the horizontal joint. At the upper edge a light steel bracing system, connected also with the internal shuttering for the two dredging shafts, held everything in position. As designed, the outer skin-shuttering was braced from the shaft-shuttering, which was very rigid. This was found to be a mistake, as the shaft shuttering was liable to be distorted during dismantling and by blows from the grabs during dredging. The bracing was consequently redesigned to hold the skin-plating true to shape independently of the shafts, but with adjustable connections to the shafts which were bolted up after the outer skin-shuttering had been trued up and checked by squaring down from a straight-edge laid across the top. No further difficulty was experienced thereafter. Lambs-tail bolts, each with a short length of distance piping and a large square nut, which was arranged to be buried in the concrete, were fixed in position through each ring of external shuttering at intervals round its periphery, their purpose being to prevent the ring from sliding up or down the concrete after the ring below it had been removed for resetting above it. Normally, two rings for each well were sufficient for routine working, but when desired three or more rings could be used. The climbing system of shuttering as used, with each ring definitely anchored to the concrete, has the advantage of keeping the building of the well definitely straight and square with the cutting-edge, without any troublesome checking by plumbing, and is a great help to accuracy of sinking. As an additional safeguard each ring was reversed end for end each time it was reset.

On the first two wells difficulties were experienced in placing the concrete. The well-steining was only 5 feet 3 inches thick, and work was obstructed by shutter-bracing and vertical suspension-rods

bedded in the middle of the wall at close spacing. It was found impossible to dispose of a 1-cubic-yard batch satisfactorily by tipping direct out of a bale-skip from the crane. The difficulty was overcome by constructing a hopper of 1 cubic yard capacity

Fig. 1.



General Arrangement of Plant for Sinking from Dry River-Bed.

with its bottom inclined at 45 degrees, and a small feeding gate and shoot discharging at a controlled rate into the work. The whole was mounted on a strong railway turntable with a steel spider bolted to its underside, and placed in position over each dredging shaft of the well alternately (*Fig. 1*). This gave very satisfactory results

as regards placing and quality of concrete, and, moreover, cut down crane-time to a minimum, as the bale-skip had a clean tip into the hopper and away. A number of these hoppers were made and their use adopted as standard practice on all wells. The concrete aggregate, being of basalt, was very heavy. It was also very angular and sharp, and although making an excellent finished concrete, was always difficult to work.

Work on the sinking of wells Nos. 1 and 2 was carried on (with an interval of 7 weeks from the end of February to the middle of April during actual floods) right through the 1931–32 flood-season, as the sites were on comparatively high ground, and were accessible from the right bank after a flood had receded, with reasonably little making-up work to the communicating embankments. At the time of the stoppage No. 1 was sunk to 50 feet and No. 2 to 40 feet below ground-level, all sinking down to this point having been through sand. Soon after recommencing work in April, at about 70 feet below ground-level, both wells got into mixed ground varying from sandy clay to very soft decomposed sandstone, and occasional beds of water-worn shingle, and sinking became more difficult. Up to this point the only obstructions to free sinking had been caused by buried trees, which were removed without much difficulty either by the grabs in the ordinary course of dredging, or after being broken up by shot-firing with small charges fixed by a diver. No. 1 well came to a standstill in this material with the sinking-effort at about 5 cwts. per square foot and the dredging 3 feet below the cutting-edge. It did not move again until the sinking-effort was raised to nearly 8 cwts. by the addition of more well-steining and 460 tons of kentledge, and with the dredging taken down 12 feet below the cutting-edge, but once it was re-started it carried on fairly freely down to within 5 feet of the specified founding-level where the sandstone became a little harder. For the last 5 feet of sinking (from 120 to 125 feet below ground-level) it was necessary to increase the weight of kentledge to 590 tons even after completing the well-steining and constructing a 10-foot length of pier. This length of pier was constructed as steining with the dredging-shafts carried up through it. Both wells were eventually founded towards the end of June, their final positions at ground level being within 2 inches of their correct positions in the up- and downstream direction and within $\frac{3}{4}$ inch transversely. The positions of the cutting-edges were correspondingly good. After founding, the wells were plugged with concrete deposited under water from bottom-door skips. The shafts were then filled with sand and the piers constructed. Both piers were finished during July.

After the completion of Nos. 1 and 2 piers the plant was moved to

wells Nos. 7 and 8, which lay on the edges of a comparatively deep channel. The curbs for Nos. 7 and 8 were built on artificial sand-banks which also carried the sinking-plant. In the meantime wells Nos. 3 and 4 had been commenced in May and June, using two more cranes which, as will be explained later, were available from the floating well-sinking sets. Wells Nos. 5 and 6 were started in September with two further cranes which became available from the same source. The progress on these four wells was much better, the aggregate sinking-time varying from 86 to 72 days as against 150 days for No. 1 and 99 days for No. 2. No. 3, without the addition of kentledge, reached full contract depth for open dredging and finished just above the sandstone bed. Nos. 4, 5, and 6 reached sandstone and refused at levels between 8 feet 6 inches and 10 feet above contract-level for open dredging, but below the maximum depth at which compressed-air sinking was called for under the contract. Before founding and plugging they were loaded with 530 tons, 656 tons, and 622 tons of kentledge respectively, but refused to move. The dredged holes were then cleaned out and plugged. Wells Nos. 7 and 8 were got down to full depth without kentledge and founded in sand in 65 and 63 days respectively, and thereafter the season's programme for this work was completed by sinking Nos. 9, 10, 11, and 12 in from 64 to 69 days each. These last four wells were sunk from a comparatively high sandbank which justified starting work on them late in the season. They were all down to safe depths before any serious rise in the river took place.

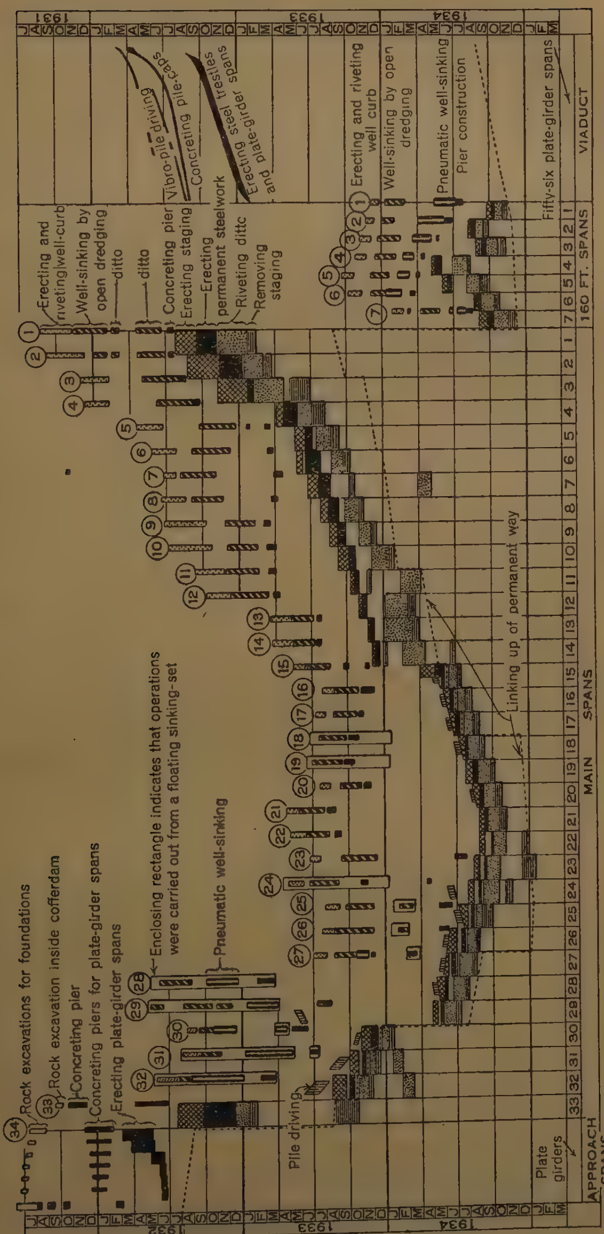
Although communications from the right bank were interrupted in January, February, and March by flood-peaks of rather more than average height, work was not delayed for long as it was possible to feed materials from the left bank by river as soon as the sites were free of water. A certain risk was accepted at these sites in leaving the heavy plant standing in the river-bed during the flood season, but in view of the level and size of the sandbank the risk was not assessed very highly. On the approach of a flood the small gear was all removed or stacked above flood-level on the well-tops. The cranes and heavy items of plant were run back to the highest point at the middle of the sandbank near well No. 12, the crane jibs being lowered and everything made fast. In the circumstances the risk was justified, more especially as it allowed of a very early start in the 1933 working-season, during which the remainder of the well-sinking was completed except for two wells, Nos. 25 and 26, where foundation-conditions were abnormal. In those two wells, and also in Nos. 23 and 27, circumstances arose which necessitated a departure from contract conditions. They will be referred to later. Of the other ten wells sunk in this season, Nos. 13 to 20 occupied from 49 to 51

days each and were founded without kentledge in sand. Nos. 21 and 22 were founded at contract-depth in clayey material overlying the sandstone rock in 58 and 54 days respectively, although in each case 687 tons of kentledge was added before they could be got down. In No. 24 the conditions were similar to those arising in Nos. 23, 25, and 26, and they will be referred to with them. No. 24 happened to be the first of the group sunk, and it was founded about 4 feet above contract-depth for open dredging under a proof load of 1,005 tons of kentledge, the sinking-time being 68 days. Three of the above wells, namely Nos. 18, 19, and 24, were commenced in water early in the season, well-sinking sets having then become available from the deep-water sinking near the left bank. The well-curbs were assembled and riveted in the sets at the end of the flood-season. The sets were then towed into position and the well-curbs lowered on to the river bed. Concreting and sinking was proceeded with until such time as the sets went aground and materials could no longer be fed to them by water transport. They were then abandoned until the water had fallen further and land communication could be established either over floating pontoon-bridges or sand embankments, when work was recommenced and carried on as in the case of dry sinking. Other well-curbs were assembled and riveted on sand islands made inside bunds in shallow water so as to be ready for sinking as soon as communication could be established. The selection of the most suitable sites for the floating-sets and the order of work for the season, which can be seen from the progress diagram (*Fig. 2*), was largely a matter of intelligent guesswork early in the season when the river was still fairly high and before its seasonal vagaries as regards formation of sandbanks and run-off channels could be predicted with any certainty. Fortunately, the guesses proved to be correct. There was water at each of the three sites where sinking-sets were installed until very late in the season. All other sites dried out or were filled in early in the season. The contractors were fortunate in having an abnormally dry river-bed in the particular season in which it could be of most service to them. The fourth floating set which was also available was held in reserve until it was clear that all remaining sites would be dried out in time.

On wells sunk by open dredging the quantities of material dredged were very rarely appreciably in excess of displacement, and there was a marked freedom from blows and their attendant troubles. Pumping, shot-firing, loading with kentledge and other expedients were only used as aids to movement when abnormal material was met with, although in sand pumping was frequently carried out as part of the process of founding.

When additional sinking-weight was required for overcoming the

Fig. 2.



Progress Chart.

resistance of harder material than normal, pumping or the application of kentledge, or both, were resorted to according to circumstances. Before pumping a well of which the cutting-edge is sealed into water-tight material such as clay, it is desirable that the penetration of the cutting-edge into the clay shall be sufficient to obviate the danger of a sudden localized blow-through from the water-bearing material overlying the clay, and the consequent unequal loosening of the material giving lateral support to the well. Two such blows occurred, but no harm was done in either case, as both wells were down near to their founding levels at the time. This limitation of the conditions in which pumping is safe somewhat detracts from its general utility, as when clay is met with it is invariably found that the maximum preponderance is required at the point where the cutting-edge is just entering the clay, and it is precisely at this point that pumping is dangerous. The system of pumping adopted in most cases was the air-lift, which is very convenient and readily applied in well-sinking conditions.

Shot-firing of small charges suspended in the water in the working chamber was found useful for the definite purpose of shaking the well and reducing skin-friction momentarily. No attempt was ever made to break down hard material from under the cutting-edges by under-water blasting, as in the Author's experience this method has never been found to be effective.

As regards the accuracy of sinking of the twenty-seven wells sunk from the right bank, mainly in the dry, only two were more than 3 inches out of position laterally, the amounts being $3\frac{15}{16}$ inches for No. 4 and $3\frac{13}{16}$ inches for No. 19. In the up- and downstream direction only two were more than 2 inches out of position, namely, 5 inches for No. 3 and $6\frac{1}{8}$ inches for No. 16. From the above results there would appear to be little room for improvement in the design of the wells as regards balance of weight and resistances and general suitability for their work when sunk in sand by open dredging, and the Engineers are to be congratulated on the success with which they made provision for the known conditions at the site. It was not suspected that clay would be met with at great depths, beyond the reasonable limits of compressed-air working. The suitability of the well-curb for meeting such altered conditions will be dealt with later.

A criticism which the Author wishes to make at this point is in connection with the finishing of the tops of the wells and founding of the piers at dead low-river level. This level was very inconvenient. If the condition is to be complied with, either exactly or with a small tolerance, it means that the depth at which the well will be founded must be predetermined either exactly or with the same tolerance before the last lift or lifts of concrete are added ; this is a condition

almost impossible in practice except perhaps in a material such as sand, or when sinking under compressed air. Further, if, as frequently happens, kentledge is required in the last stages and it is doubtful to what depth the well will sink, it will almost certainly have to be removed at least once and probably oftener for the addition of extra concrete before the well is founded, and replaced after allowing time for the concrete to harden. The sinking-weights are therefore limited just when they are most necessary and time is lost when it can be least afforded. Again, except at dead low river, the well-head cannot be kept dry for the handling of kentledge nor the pier commenced except within an inconvenient and probably expensive cofferdam. The difficulty was evidently recognized by the Engineers when they included a contract clause permitting, with certain reservations, the addition of a limited length of pier to a well if necessary before founding and the carrying up of the dredging-shafts through this length of pier. It is submitted that that is not enough. Unless there is some special reason to prevent it, the well proper should be carried up to give a working freeboard when founding above at least the highest expected river-level during the working season.

River Regime.

On a falling river for several months after a flood-season the river-bed is extremely unstable. It is nearly impossible until June or July to predict the positions of the sandbanks and deep low-river channels which will be formed. During the unstable state of the river very little suspended matter is carried by the water, and for practical purposes all bed-movements are due to the rolling of particles of sand along the bottom. The critical water-velocity above which rolling takes place appears to be between $1\frac{1}{2}$ and 2 feet per second. Whenever the bottom speed exceeds this amount, sand from the bottom and sides is displaced, more especially by side scour and undercutting against steep banks. The sand so displaced is rolled along the bottom until it comes to a place where the local channel down which it is travelling widens out. Here it is rolled up a very flat gradient and fans out in the form of a submerged delta, and the channel generally bifurcates. At its downstream margin the delta ends in a steep bank falling back into deep water at a slope of approximately 1 to 1. As more and more sand is rolled up the gradient and over the lip, the downstream edge of the bank moves downstream very rapidly. Eventually the whole bank may move downstream by displacement of sand from the flat gradient and building up below the downstream lip. Rates of movement or building-up of the order of 20 feet per day on a bank 10 feet

deep have been observed. So long as the river-bed into which the bank is advancing remains wide and approximately uniform in depth, the bank continues to fan out and advance at a gradually decreasing rate as the length of the lip increases, and remarkable uniformity of spreading is maintained, the shallow depth and velocity at the crest presumably adjusting themselves to keep the sand on the move. Movement in the downstream direction is eventually arrested either by a cut-off channel due to outside influences coming in at an angle and carrying away the sand as it comes over the lip, or by a fall in the river reducing the quantity of water passing over the crest so that velocities fall below the critical velocity.

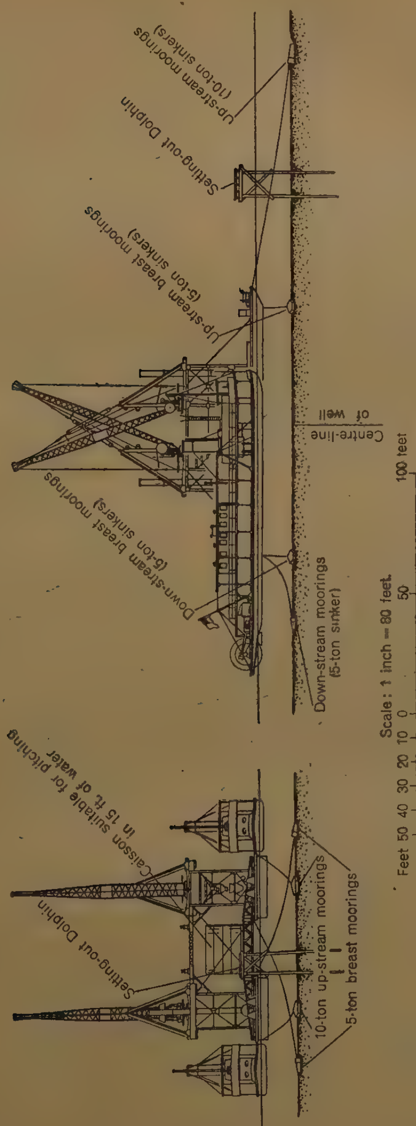
Description of Work.

The floating well-sinking sets, the general arrangement of which is shown in *Figs. 3*, were ready for use at the beginning of the 1932 working-season. The working-moorings decided on for use during sinking operations consisted of wire ropes attached to heavy concrete or cast-iron sinkers. Normally two 10-ton sinkers were laid out upstream and four 5-ton sinkers for breast moorings. The breast mooring-wires, made fast near to the up- and downstream ends of the set, were crossed on the centre line of the set below the level of the barge-bottoms so as to be clear of craft brought up alongside the sets. In other words, the wires from the port-side barges were attached to sinkers laid out on the starboard side and vice versâ. One 5-ton sinker was generally laid out downstream. All wires were led inboard to the bitts where they were made fast, and accurate adjustments of position were made by means of tackles attached to the wires by stoppers and having their falls led to 2-ton hand crab-winches. Sinking-set positions were controlled from instrument-platforms on 4-piled dolphins erected about 80 feet upstream of the bridge centre-line and 6 feet to one side of the major axis of the well to be sunk.

In view of the uncertainty as to sites and depths of water in which caissons would be pitched, two of the first three caissons were built up to a height of 20 feet above the cutting-edge, which was the maximum height for building under the jacks before lowering, and the third was built up to 16 feet. It was intended that one caisson of each height should be pitched at sites where the water would be about 12 feet and 10 feet in depth respectively. The third caisson was intended to be built up further to suit the site of well No. 32.

The flood-season was late, and the river-levels were too high and the bottom-conditions much too uncertain to commence work at

Figs. 3.



FLOATING WELL-SINKING SET.

any time in April or during the early part of May. About the middle of May there was 12 feet of water at No. 28 and 10 feet at No. 29. The water at the site of No. 30 was somewhat shallower. From the point of view of current-velocities early in the season, the farther

out from the left bank the first wells could be pitched the better, but they should not be pitched so far out as to incur serious risk of discontinuity for steelwork-erection purposes from the left bank in the following season. Wells Nos. 28 and 29 were selected for the first two sinks. At No. 32 the conditions were exceptional. It was obvious that this well would be troublesome owing to uncertain depths, high river-velocities early in the season, and the small cover of sand over a fairly steeply-sloping rock surface. At the same time it was essential that the well should be started early so as to be completed in the 1932 season.

The fourth set was in reserve to start work at either No. 30 or 31, and it was anticipated that at least one of the first three sets would be available for a second sink towards the end of the season in time to take on the remaining well if the site did not dry out; it might even be possible to employ it on No. 27 if either No. 30 or No. 31 could be pitched in the dry.

During May the first three sets were got into position. As soon as they were in position they caused disturbance in the bottom-conditions at the sites, and the resultant changes were accentuated when the cutting edges of the caissons were lowered into the water. In view of these changes it was decided that an extra strake of skin-plating would be required on No. 29. This caused a short postponement of the date at which this caisson could be pitched. The caissons at both No. 28 and 29 were got on to the bottom early in June, and the jacking links were let go after putting in concrete up to the level of the top of the cone-plating, 8 feet above the cutting-edge; the caissons were then sunk from 6 to 8 feet into the river-bed. Meanwhile the river was falling, and as soon as No. 29 was well entered the river-bed commenced to rise up around it. Work on it had to be interrupted almost immediately, as the sinking-set grounded and access by water transport became impossible. At well No. 28 conditions were better, and by the end of June the cutting-edge had been sunk 24 feet into the river-bed and the concrete steining carried up to 40 feet above the cutting-edge. Sinking on this well proceeded normally thereafter until the cutting-edge reached stiff red sandy clay 86 feet below the river-bed level at the end of July. From this depth it was sunk under compressed air to founding-level.

Pitching Caisson for Well No. 32.

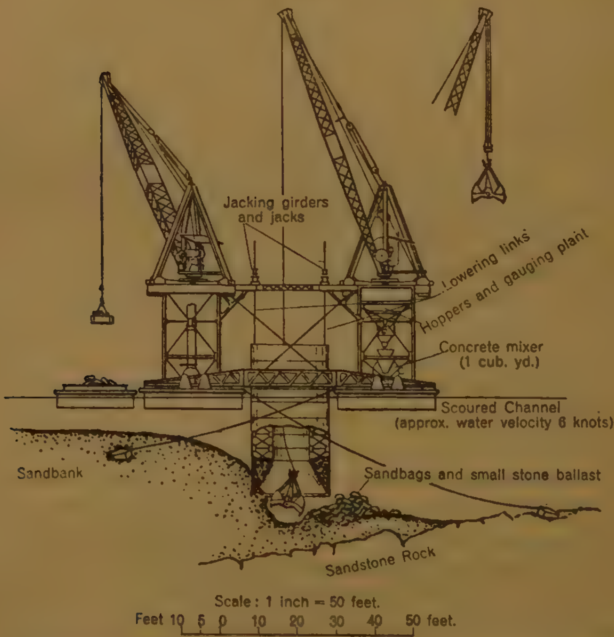
At the site of well No. 32, where the third set had been got into position by the end of May, the water was deep and running strongly at about 4½ or 5 knots. The direction of the current was not normal to the bridge centre-line but had a strong side-set caused by the rocky promontory on which pier No. 33 was built. The general

direction of flow was inclined at 30 degrees or more to the normal. The rock surface which was just exposed at dead low river at pier No. 33 was about from 29 to 33 feet below low-river level at well No. 32. Water-level, which was 9 feet above low-river level at this date, was about 38 feet above the highest point of the rock. The caisson was lowered into the water and the building-on and riveting of extra strakes commenced. During these operations great care had to be exercised not to overload the jacking gear, which was only capable of handling about 90 or 100 tons of unbalanced weight with safety. The buoyancy and depth of immersion were regulated accordingly by lowering and by the addition from time to time of concrete filling. The total weight in air soon became such that it would have been impossible to lift clear of the water again without dismantling. By the end of July the skin-plating had been built up to 38 feet above the cutting-edge, which was suspended 16 feet below water-level, soundings of 28 and 30 feet having been observed in the interval. Conditions at the site up to this time had never been sufficiently stable to justify the risk of pitching the well, which would probably have grounded on bare sloping rock.

Early in August the river-bed was showing signs of building up a little, but some anxiety was felt on account of the approach downstream of a sandbank which appeared likely to extend around the sites of Nos. 30 and 31 and near to the side of No. 32, leaving the site on a steep side slope of sand between the sandbank and the deep channel. It was decided that the caisson must be grounded or at any rate suspended at as low a level as possible without further delay. On 8 August the depth of water varied from 17 feet on the offshore side to 24 feet on the inshore side and 24 feet also at the up- and downstream ends. Lowering was commenced but had to be carried out by stages whilst concrete was added in small lifts to reduce buoyancy. The balance of weight carried by the jacks was not sufficient to lower quickly through more than a few feet and land on the bottom with a good working preponderance. As lowering proceeded, the bottom deepened all round and by the 10th August varied from 20 feet on the offshore side to 30 feet on the inshore side with a pothole 33 feet deep. The cutting-edge was 20 feet below low-water level and just touching the bottom at one side. The weight on the jacks was about 50 tons, which limited the possible lift to about 15 inches. The lip of the sandbank was approaching very close on the offshore side and the side slope steepening continually. Dredging was started, and concreting, dredging and lowering carried on continuously. At the same time barge-loads of sandbags and small stone were dumped as rapidly as possible on the deep side, but in spite of this, as lowering proceeded the cutting-edge was

pushed over about 4 inches towards the shore, much of the stone and some of the sandbags being carried away or undercut by the current. By the morning of the 11th August the cutting-edge was 25 feet below water-level and the river-bed on the offshore side had piled up again to 18 feet below water-level, whilst the cutting-edge was still unsealed and clear of the bottom on the inshore side. The lateral movement had increased to 16 inches. By the morning of 12 August the cutting-edge was down to 30 feet and the upstream

Fig. 4.



CAISSON DURING LOWERING OPERATIONS AT WELL No. 32.

end sealed. Further depositing of material outside and inside completed the seal and secured the caisson, which was, however, 16 inches too far inshore at the upstream end and 18 inches too far inshore at the downstream end. In the up- and downstream direction the position was correct. After sealing, concreting was proceeded with without delay in order to get good preponderance on the bottom, and as soon as possible thereafter compressed-air gear was installed and sinking commenced under air. The pitching of this caisson is shown in *Fig. 4.*

The position would undoubtedly have been worse but for the

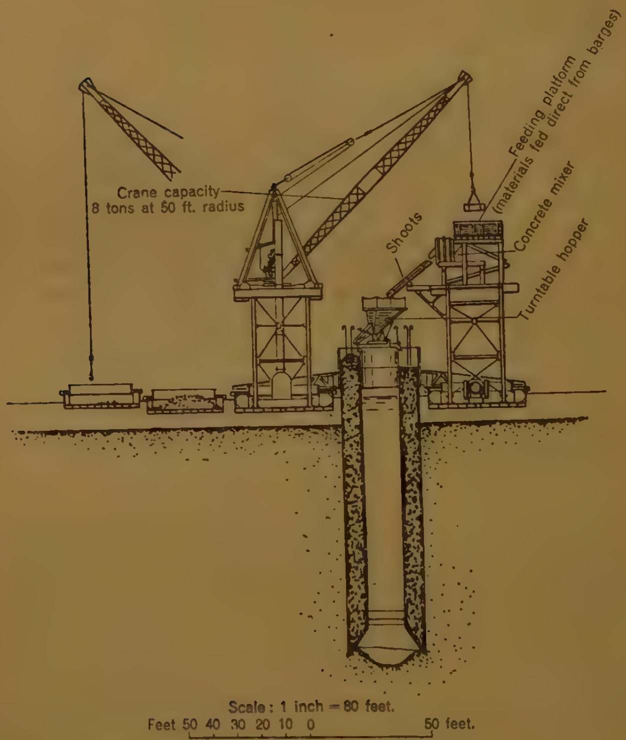
very large righting-moment of the caisson with the cutting-edge 25 feet below water-level and only 9 feet of concrete above the cutting-edge. If the caisson could have been lowered nearly on to the bottom whilst the water was still deep it might have been possible to build up the bottom around it later, but it should be borne in mind that the current was very strong and setting in a difficult direction. As things were in the emergency it would have been of great assistance if the jacking gear and plant generally had been capable of dealing with a much heavier load so that lowering through a considerable depth could have been carried out quickly. Additional buoyancy during lowering could have been obtained by putting on the domes and applying compressed air, but this procedure would have had the disadvantage of blocking the dredging-shafts, and could not therefore be considered. It would also have reduced the righting moment, and any misjudgement of air-arrangements might have caused disastrous overloading of the jacks. It should be remembered that air-pressure could not have been released until all the jacking links had been let go. Before sinking was commenced the well was tilted in an endeavour to correct the position of the cutting-edge to some extent before meeting the rock. It was also anticipated that it might be possible under air to work it over from the high side of the rock excavation, but by that time the sand on the offshore side had banked up higher and become more compact, and it was not possible to make any appreciable correction. This was not a serious matter, as the offset at well-top level provided for a correction of up to 21 inches.

In the 1933 season, as has already been mentioned, three of the sets were used for pitching curbs at wells Nos. 18, 19 and 24, whilst the fourth was held in reserve ready to deal with any remaining site where the work could not be undertaken in the dry. Several of the sets were again used, after dismantling the superstructure and removing the downstream cross booms, as floating platforms from which to regulate the deposition of pitching stone around the piers in deep water. As designed, the sets had two cranes each, one being intended for off-loading materials and charging the hoppers and the second for lifting concrete from the mixers into the work. It was early realized that, in addition to saving one handling and releasing half the cranes for work in the dry river-bed, better routine-working could be achieved by putting the mixers on top of the superstructures with gravity discharge into the turntable-hoppers on the well-heads, and feeding them directly batch by batch from the barges alongside on to a small working platform above the mixer-hoppers. This alteration was made and proved eminently satisfactory (*Fig. 5*, p. 396).

Pneumatic Sinking.

The difficulties and risks inseparable from work carried out under compressed air are accentuated when the work is undertaken in a hot climate and at high pressures such as were called for in the Lower Zambezi bridge foundations. They become still more formidable when the circumstances of the work necessitate the em-

Fig. 5.



FLOATING WELL-SINKING SET CONVERTED FOR WORKING WITH ONE CRANE.

ployment of raw native labour speaking only primitive languages and dialects and having no sort of understanding of the reasons for the elaborate precautionary measures which are necessary. Natives were at first very reluctant to go through the locks, but that difficulty was soon overcome and once they were accustomed to the novelty they rather enjoyed it than otherwise. They stood up to the pressures, which at times exceeded 50 lbs. per square inch, very well indeed. A more serious difficulty was to find sufficient local

supervision to train for the work, with the necessary qualifications as to willingness, intelligence, physical fitness, and capacity in the handling of labour. It was quite out of the question to obtain men experienced in compressed-air work locally, as none at all were available, and it was equally impossible, on grounds of expense and the general undesirability of bringing out numbers of men on short engagements, to obtain them from Great Britain. The pressures in caisson No. 32 were comparatively light, and advantage was taken of this fact to train labour and overseers there when possible before putting them into very high pressures in some of the other wells.

Decompression times and lengths of working shift were regulated according to pressures but were varied as and when necessary. As a rough generalization it may be said that, up to a maximum of from 35 to 40 minutes, 1 minute per pound of working pressure was allowed after a full working shift. The basic lengths of shifts were 8 hours up to 35 lbs. per square inch, 6 hours to 4 hours between 35 and 40 lbs., and 2 hours twice in 24 hours at over 40 lbs., but these times were varied as between one well and another. All compressed-air personnel, both European and African, were medically examined before acceptance for the work and kept under special observation thereafter. The natives had special rations and care was taken to avoid danger from chills. All cases of "bends" were put back under pressure immediately and treated. The percentage incidence of bends is interesting, being a maximum for both Europeans and natives at about 35 lbs. pressure. The overall average incidence is lower for natives at 0·43 than for Europeans at 1·2 cases per hundred decompressions. Out of a total of six hundred and fifty-six medical-lock treatments of natives, twenty received subsequent treatment in hospital and five died. The corresponding figures for European overseers were seventy-three medical-lock treatments, fifteen hospital cases and no deaths.

The soft sandstone removed under compressed air was a difficult material. It was too soft for the satisfactory use of explosives and too hard to excavate by hand without heavy physical effort. Pneumatic tools were provided but were of little service and the best progress was made by hand labour with hammer and wedge. Excavation and poling back under the cutting-edge were rendered extremely difficult by the blunt edges specified on all wells where it was known that rock would be encountered. The Author would have much preferred a sharp cutting-edge around the periphery with a wide blunt finish to the cross wall at a level several feet higher than the cutting-edge proper. The special domes called for under the contract were interesting, their purpose being to render it possible to apply air conveniently at any time to any well being sunk, and,

if desired, to revert to open dredging and again to compressed air at will.

Sinking into Plastic or Cohesive Material.

All the wells from No. 21 to No. 28 inclusive, before reaching contract founding-depths, encountered either clayey material varying from clayey silt to a good stiff grey clay, a muddy black clay locally known as "m'tope," or a reddish material of varying consistency having the general appearance of decomposed sandstone. In all cases the wells gave trouble, and, except in the case of No. 23 where the material first encountered was silt, it became necessary to surcharge all the wells heavily with kentledge before penetration into the material could be obtained by open dredging. In No. 28, as it was known that rock was present at above the limiting depth for compressed-air working, the excavation of the material was left to be done under air after the well had refused by open dredging under 400 tons of kentledge. On No. 27 compressed air was also applied, the material being similar to No. 28, but in this case the well was founded in the material which, for want of a better name, may be described as a fairly hard decomposed sandstone, a few feet below "limiting" air depths but before reaching solid rock. The remaining wells were got down to contract-depths under varying loadings of kentledge, namely 687, 687, 1005, 930, and 930 tons respectively, combined with various other expedients, such as pumping out the wells to a maximum depth of as much as 40 feet below river-level to increase effective weights, shot-firing to shake the well and release skin-friction on the surfaces in contact with sand, and in several cases undercutting to break down the bearing resistance of the material under the cutting-edge. In attempting to keep the offsets accurately at contract-level and at the same time to keep the concrete above water, the whole of the kentledge had to be removed and replaced, after adding concrete and waiting for it to harden, in most cases once and in some cases more than once on the same well. At Nos. 21, 22, and 24 the material at contract-depth was accepted for founding. At Nos. 23, 25, and 26 the material was not acceptable, and work was suspended pending a decision as to what steps should be taken. This was particularly unfortunate, as they were the last three wells and the stoppage occurred very near the end of the working season. It was eventually decided to carry on sinking, and No. 23 was sunk at once to an acceptable bottom at 11 feet below contract-depth. The decision came too late to permit of finishing Nos. 25 and 26 before the rise of the river. The plant and 1,000 tons of kentledge had to be removed from the river-bed after adding extra concrete to bring the well-heads above normal high-

river level. Work was suspended altogether for 3 months and then resumed from floating well-sinking sets. Fortunately, it was possible to take advantage of the absence of high floods and to get both wells founded, at 14 feet and 4 feet below contract-level respectively, during March. Only a slight interference with the progress of steelwork-erection was thus caused.

Wells Sunk for Secondary Spans.

As well-sinking and compressed-air plant became available it was transferred to the wells of the 165-foot spans which had been ordered in November, 1932, in substitution for a portion of the right approach-viaduct. Open sinking on these wells proceeded with absolute regularity down to soft sandstone, and the compressed-air work, which was limited (on account of shortage of local supervision) to actual sinking on one well only at a time, with spare locks for changing over, showed very satisfactory results, the overall time worked under air being 250 days during which an aggregate of 120 feet in depth, or an average of nearly 6 inches per well per day, of sinking in soft sandstone rock was carried out on the seven wells. The penetrations into sandstone varied from 5 feet 4 inches on a well sunk 107 feet to 28 feet on a well sunk 95 feet below ground-level.

Sinking-Effort and Resistances.

In well-sinking operations resistance to sinking is attributable to two main factors :—

(1) Skin-frictional resistance of the material through which the well is sunk acting on the surface of the well in contact with that material.

(2) End resistance of the material into which the cutting-edge has penetrated.

The forces tending to produce sinking are commonly assessed in units of "sinking-effort"; that is to say, the total effective sinking weight (after deduction of the buoyancy) divided by the surface area of the well below ground-level. The minimum sinking-effort of any normal well occurs when the top of the well-steining is flush with the ground and the water-level is also at ground-level. The sinking-effort produced in these conditions by a well of any given design is for all practical purposes constant at all depths of cutting-edge, the only variation being due to the weight of the well-curb. At or near the designed depth it may be regarded as a definite constant for the particular design, with concrete of a definite weight. During the process of sinking, when part of the well is nearly always above ground-level and the water-level varies in relation to ground-level,

the sinking-effort is always higher than this figure until the final stages, and if the sinking-effort constant is greater than the mean skin-friction the well has always a definite preponderance, which will cause movement as soon as the end resistance is removed.

In the wells of the Lower Zambezi bridge the sinking-effort constant was 4·67 cwts. per square foot of well-surface, and in all cases of wells founded in sand (in many of which the top of the steining was at ground-level and very near to water-level) the wells went down to founding depth without any trouble and without the addition of any kentledge, the only indication that a greater proportion of total available sinking weight was being absorbed by skin-friction being that wells did not normally drop in the final stages until the sand had been excavated about 3 feet below the cutting-edge in the dredged hole as against 1 or 2 feet above in the earlier stages. The Author does not necessarily wish to imply that the unit skin-frictional resistance was increasing. Sinking-efforts were definitely less as founding-depths were approached and well-heads reached ground-level. In nearly all cases pumps were put on for founding, but the purpose of this pumping was to get the cutting-edge bedded as far as possible into undisturbed ground without dredging more than was necessary below the cutting-edge, and these wells could have been got down by dredging only.

In these conditions an analysis of sinking-efforts during the process of sinking would have no meaning, the relation of the figures being at all times subject to and controlled by fortuitous end resistances of indeterminate magnitude. It is, however, clear that at these depths the skin-frictional component resisting sinking was less than 4·67 cwts. per square foot in all cases in which the whole well was in sand. The relationship between effective sinking-weights, depths of dredging, and movements of the well can be seen in the typical sinking-diagram shown in Fig. 6, Plate 1. In the Author's opinion the skin-frictional resistance probably amounted to about 4·25 cwts.; this is borne out very closely by the actual figure at which one well (No. 27), under compressed air, moved when the air-pressure was lowered and the whole of the cutting-edge was quite free. In this case the depth was similar and practically the whole of the surface exposed to skin-friction was in sand. This value is in very close agreement with the "sinking-effort" value for this depth given by Sir Robert Gales in his Papers on the Curzon [1] and Hardinge [2] bridges where the wells were sunk through sand and the residual end resistance was presumably negligible.

[1] Minutes of Proceedings Inst. C.E., vol. ccv (1917–18, Part I), p. 18.
[2] *Ibid.*, vol. clxxiv (1907–08, Part IV), p. 1.

Whenever the cutting-edge of the Zambezi-bridge wells came into contact with plastic material such as clay the resistance to movement immediately increased to a very marked extent. The dredged holes in these circumstances generally went down to from 10 to 17 feet (in some cases even more) below the level of the cutting-edge (a proceeding not to be recommended unless the cutting-edge is thoroughly sealed), before movement took place, even with the effective weight enormously increased by the addition of either extra well or pier concrete, kentledge, or both.

It was very noticeable that, apart from increased loading, the most difficult stage of sinking in which to produce movement occurred when the cutting-edge was in sand a few inches above the clay. A similar sudden increase of resistance in similar conditions appears to have been observed at the Willingdon bridge.[1] This is no doubt accounted for by the fact that at this stage full wedge-action could only be gradually developed by the cone-plates and also that the narrow flat bearing area at the junction of the cone-plates and cutting-edge was increased to a larger effective bearing area by the spreading or ballasting action of the sand overlying the clay. After the cutting-edge had entered the clay, movement usually took place more freely, but even so, the loading necessary to produce it was very high. In all cases where clay was encountered a comparatively small proportion only of the surface of the well exposed to skin-friction was in clay, and from the evidence of the wells sunk entirely in sand it seems safe to say that at the very most not more than 4·5 cwts. per square foot on the well area could have been due to skin-friction, and that the end resistance could not therefore be less than the balance of the total effective weight applied. Analysed in this way the full loads applied in these cases to overcome end resistance as distinct from skin-friction were of the order of 1,300 tons, which is equivalent to 11 tons per linear foot of cutting-edge including the cutting-edge of the cross wall. These heavy loadings to overcome end resistance appear to call for investigation. Assuming that a long straight vertical face of clay exists 10 feet high, with a long 1-in-$2\frac{1}{2}$ wedge acting on a line 5 feet 3 inches behind the top edge of the face and parallel to it, it seems inconceivable that movement should not take place long before 11 tons per linear foot of cutting-edge of wedge had been applied. The actual conditions in practice were obviously very different from a straight-line wedge, much of the cutting-edge of the wedge being circular in plan and only 20 feet in diameter, so that the material had to be forced into a hole con-

---

[1] Robert Mair, "The Willingdon Bridge, Calcutta." Minutes of Proceedings Inst. C.E., vol. 235 (1932–33, Part I), p. 36.

26

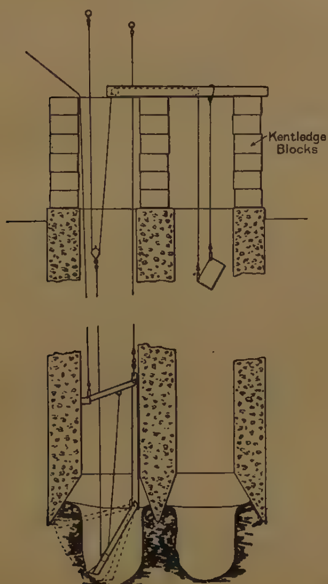
centric with the cutting-edge and only about 8 or 9 feet in diameter. The wedge was, in fact, operating against a very strong horizontal circular arch of clay at least 5 feet thick and spanning a hole probably not more than 8 feet across. Movement could hardly take place until the internal resistance of the arch material had been at least partially broken down and the whole body of the clay had suffered plastic deformation. The evident conclusion is that the normal design of well-curb, whilst eminently suited for work in a granular material such as sand, is by no means ideal, and should be capable of improvement for sinking in clay or other cohesive or hard material. Where such material may be expected during any part of the sinking, and more especially near founding-depths, very careful attention should be directed to the design of the cutting-edge and to the provision of methods for reducing end resistance. The mere provision of more weight cannot be regarded as a satisfactory final solution of the problem. The main requirements for such work would appear to be :—

(1) that the total working length of the cutting-edge should be kept as short as possible ;
(2) that the horizontal bearing surface of the underside of the cutting-edge proper should be reduced to as small an area as possible (consistent with strength to avoid buckling up if it should come into contact with hard rock or other obstruction) ;
(3) that as much as possible of the wedging action should be applied on straight lines in plan instead of on lines concave towards the free face ;
(4) that the design should permit of dredging out a space of such minimum cubic capacity and shape that it will receive the body of material forced off by the wedge in its natural shape without deformation. It is self-evident that the larger the dredged hole and the nearer it can be got to the cutting-edge the better.

It would seem that these requirements could be more nearly met than they are without loss of efficiency in other directions. Cutting-edges with less bearing resistance than those used on the Zambezi bridge could easily be constructed. Cross cutting-edges could be omitted and the cross wall started at a higher level. End walls and cross walls could be kept much thinner, at any rate for the bottom 25 feet or so, which would enable the grabs to be slued under the corbels so formed and to clear out a practically continuous trench from end to end of the well-curb, leaving only the side walls of clay to be broken down by wedge-action.

The circumstances of the work, in which clay was only met with at considerable depths and the total quantity dealt with was comparatively small, did not render it possible to do very much by way of experimental confirmation of the end resistance due to the horizontal arching effect, but an undercutting tool was devised and constructed with the object of breaking down the arch (*Fig.* 7). This tool was installed in two cases and in both had the immediate effect of freeing a well which had come to a standstill. The assessment of reduction of end resistance was necessarily inconclusive,

Fig. 7.



but some idea of its importance may be formed from the fact that whereas a well under kentledge had failed to move when the internal water was lowered 35 feet by pumping, it did move freely after six radial cuts or slices had been taken with the tool with a lowering of the internal water of only 17 feet. No other operation had been performed in the interval. Ignoring the possible lubricating effect of pumping, the seal being nearly perfect at the time, and also the greater theoretical lateral support which the clay should derive from the pressure of the water at the higher water-level, the theoretical difference in buoyancy at the two different heads of water was 250 tons, so that it appears probable that even the very incomplete break-

ing down of the arching effect by the tool did succeed in lowering the end resistance by at least 250 tons, and probably more.

VIADUCT: PILE-DRIVING.

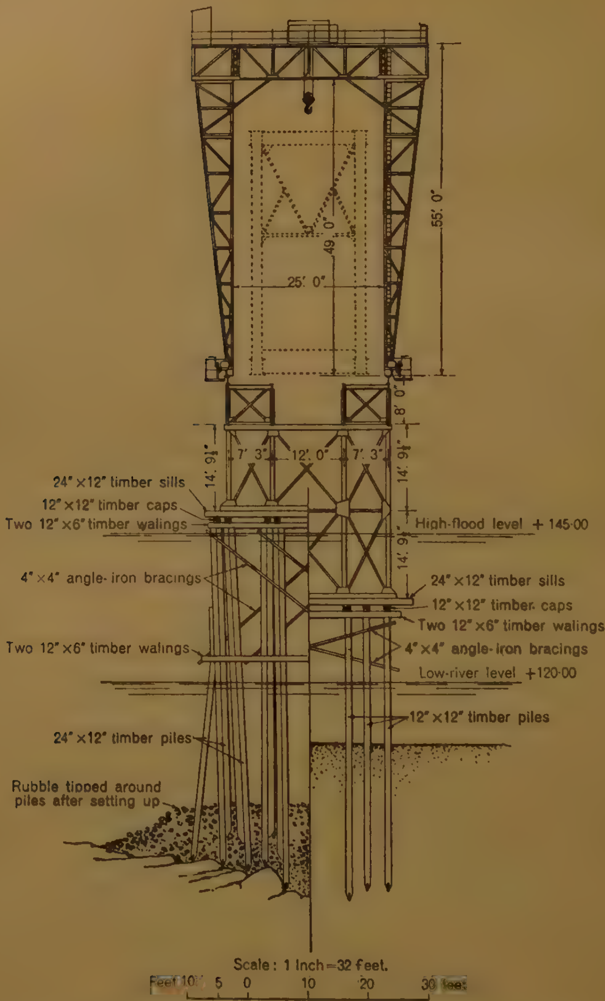
Pile-driving for the right approach-viaduct was commenced in March, 1932. "Vibro" piles 17 inches in diameter were called for, and the normal special plant required for this process was used. A minimum length of 30 feet was specified and a final set of not more than 1·5 inch for ten blows during the last 4 feet of driving was required as a condition of acceptance. It soon became evident that neither condition could be complied with, as soft sandstone rock, which had been diagnosed from the borings as sandy clay, was met with at gradually decreasing depths reaching a minimum of about 10 feet below ground-level at bent No. 13. The severity of final sets was increased in an endeavour to penetrate into the sandstone, until the gear gave way under excessive punishment and a pile-tube was crippled. At this time sets of under ½ inch for ten blows of a 2½-ton hammer with a 3-foot 6-inch drop were being insisted upon and the driving force developed must have been of the order of 200 tons. Work was discontinued in the middle of April, and it was not until the beginning of June that it was agreed that piles shorter than 30 feet would be accepted and that the requirement of 4 feet penetration at minimum set was abandoned. The very severe ½-inch set for ten blows was retained for all piles less than 30 feet long and it was necessary to proceed with extreme caution. In spite of every precaution, however, another pile-tube was crippled and a number of shoes broken, rendering the piles useless. After bent No. 24 the depths at which sandstone was met again became greater than 30 feet and driving conditions reverted to normal. The work was again stopped at bent No. 50 at the end of August, three hundred and sixty-six piles having been driven up to this date. Later, when the decision to substitute extra spans for viaduct was made, the piling was completed up to bent No. 56. Originally, the contract provided for the driving of "extension piles" by a special process when the specified set was not obtained within the limits obtainable with a 55-foot pile-tube, but this process was never tried on the works under review. For normal conditions on this type of work the "Vibro" process has many advantages from the contractors' point of view. The special gear has been very carefully thought out and is easily handled. Very good progress can be made and there is no reason to doubt the soundness of the results. The failures of the gear which occurred under altogether exceptional conditions are in the Author's opinion in no way derogatory. A

precast pile would have made no better showing under the same punishment. Reinforced-concrete pile-capping and the erection of steel trestles and plate-girder spans followed the pile-driving, but this work also was stopped at span No. 49 in conformity with the stoppage of the pile-driving.

Steelwork-Erection.

At the end of July, 1932, work was begun on the erection of the temporary steel staging and the goliath cranes in spans Nos. 1 and 33 simultaneously. The erection of permanent steelwork was commenced in both spans at the end of September. The general design of the temporary staging and the method of working are shown in *Figs. 8* (p. 406) and Figs. 9, Plate 1. In span No. 33 the full height of steel bents was used to support the temporary girderwork, the exposed sandstone rock of the river-bed being levelled up to the underside of the temporary sills by tipping mounds of quarry waste. In span No. 1, where the height of staging was too small to employ even the half height of steel bents, 12-inch by 12-inch timber trestlework was used to support the temporary girder-work. In span No. 33 materials were fed over the permanent railway track laid on the completed left-bank approach-spans. In span No. 1 materials had to be fed from the ground owing to the discontinuity between the completed part of the right approach-viaduct and the commencement of the main spans, all work in this section having been held up pending the decision as to substitution of wells and spans for a portion of the piled viaduct provided for in the original contract. This method of feeding necessitated the releasing of one of the 10-ton steam derrick-cranes from work on well-sinking and also caused some congestion on the main service track from Sena yard down to the steelwork picking-up point, as all materials and plant for well-sinking, steelwork-erection and for stone-pitching around the piers on this portion of the bridge had to be passed over it. A temporary earthwork ramp was constructed so as to ensure that communication with the steelwork-erection point would not be cut off suddenly by an unexpectedly early rise in the river. It was not, however, of much use except as an emergency safeguard owing to the steepness of the gradient and the unavoidably sharp curves necessary to keep as clear as possible of the alignment where the method of construction was still in doubt. Permanent steelwork on the right bank was carried on until stopped by floods at span No. 3. The work was, however, secured against accident without difficulty, communication being maintained over the earthwork ramp. Staging-erection was recommenced at span No. 4 in April and all the spans from this end up to and

Figs. 8.



SPECIAL PILING AND FALSEWORK FOR SPAN NO. 32.

NORMAL PILING AND FALSEWORK.

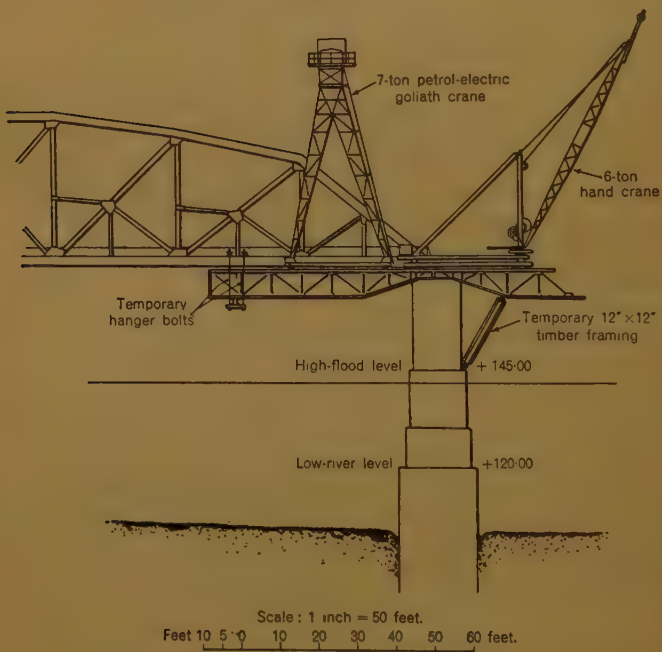
including No. 14 were erected before the end of December, 1933.

On the left bank only one span could be erected in 1932 as work on well-sinking and the construction of pier No. 32 was not completed. The same river conditions which had delayed the com-

mencement of well No. 32 in the previous working-season again caused serious trouble and anxiety in the early working-season of 1933 in connection with the staging of span No. 32. The river-bed consisted almost entirely of bare rock with a very uneven surface. The mean side slope was about 1 in 6, but there were vertical-faced ledges and crevices and steep slopes up to 6 or 7 feet in depth in many places. Diving work was impracticable owing to the strong local current running up to at least $5\frac{1}{2}$ knots through this span early in the season. Piles could not be driven sufficiently far into the sandstone to support themselves until bracing could be fixed. A light steel cage was constructed on a small pontoon of 10 feet beam which was moored securely in position with upstream and downstream and breast moorings. The pile-frame was brought alongside on separate floats and a complete bent of 24-inch by 12-inch timber piles was driven as far as possible into the sandstone on each side of the cage pontoon, care being taken to ensure that the weights of the piles were sufficient to overcome buoyancy. Each pile when driven was attached to false leaders on the cage by a toggle bolt, allowing free vertical movement of the cage, before the pile-frame was let go. Caps, low-water walings and bracings above water were fixed before the cage pontoon was withdrawn. The 24-inch by 12-inch piles were sufficiently long to reach about 15 feet above water-level for bracing purposes, and the completed structure was similar to a table with legs unbraced for 30 to 35 feet below water-level standing on the bare rock bottom. Once the above-water bracings were completed, quarry waste was tipped round the legs to a depth of about 10 feet above the bottom. The steel staging bents were then erected on top of the pile capping and the erection girders put on in the ordinary way. The height from the deepest rock surface to rail-level of the goliath crane was 87 feet and to the working-platform of the goliath 143 feet. The distance, centre to centre, from the extreme upstream to the extreme downstream piles was 30 feet. The staging was tested with a 25 per cent. overload before the erection of permanent steelwork was commenced. Owing to these difficulties, erection of permanent steelwork did not commence in this span until the 30th September, 1933. The span was, however, completely assembled 8 days later. Only two more spans, Nos. 31 and 30, were erected at this end of the bridge in 1933. In these spans, as in all subsequent spans, the depth of sand overlying the rock was quite sufficient to give a good hold for the staging piles. Work at both ends was suspended in December, 1933, as it was not considered safe to place any reliance on piled staging in the main channels during the flood-season. The disposition of plant during the flood season is shown in *Fig. 10* (p. 408).

Work on pile-driving and staging was recommenced at both ends in April, 1934, in spans Nos. 15 and 29, and permanent steelwork at both points near the middle of May. The last of the main spans (No. 22) was fully assembled by the first week in October, thus completing the erection of the last fifteen main spans in under 5 months. During this season also the seven 165-foot spans were all erected, the last of these being completely assembled and the bridge joined up by the middle of October. Span-erection progress can be clearly seen from *Fig. 2* (p. 387). The best assembling time

Fig. 10.



for any one span was 4½ days. No work was done on span-erection or staging by artificial light.

Advantage was taken in a few spans of the contract-provision for drifting up a span with 60 per cent. drifts and 40 per cent. erection-bolts in order to release staging before riveting had been completed, but even so the overall rate of progress on span-erection was always dependent on staging-time and the goliaths were only at work for approximately 40 per cent. of their possible time. The fabrication of the permanent steelwork was extremely accurate. All members were found in practice to be absolutely interchangeable one span

with another, and no trouble was ever experienced either in making a joint or in removing drifts on the temporarily-drifted spans. This result was largely due to the use of steel-bushed jigs throughout during fabrication in accordance with the provisions of the contract. Only the first two and the last two of the thirty-three main spans were assembled at the works prior to shipment, the assembly of the last spans being solely for the purpose of inspection for estimating the results of wear on the jigs after use on so many spans. The first train was run over the bridge on permanent track on the 24th January, 1935, and the full regular train service commenced on the 1st March, 1935.

Stone-Pitching Around Piers.

Stone-pitching around the main piers was not completed until later owing to the difficulty of obtaining a sufficient quantity of basalt of sizes acceptable for pitching stone. Whenever the water was less than 7 feet deep (the thickness of stone to be deposited), the stone was tipped direct from 10-cubic-yard side-tip wagons on the railway track on the bridge down four temporary shoots at each pier, and afterwards spread to correct thickness by hand. Where the water was more than 7 feet deep stone was transported by water, and depositing was carefully regulated from the downstream edge of a floating arrangement of coupled barges covering the whole width of the stone-pitching around the pier. Depositing was commenced at the downstream side of the pier and the barges gradually warped upstream as required.

Conclusion.

From a purely technical point of view the construction of the Lower Zambezi bridge did not present many features of great novelty. Much of the interest lay in the opportunities offered for systematic improvement in working methods, from the point of view of economy and time, by the unusual number of repetitions of the same cycles of operations. The vagaries of seasonal river-bed movements necessitated a working programme sufficiently flexible to allow of considerable alterations in detail at very short notice. The main problems, apart from stone-supply, were essentially transport problems involving the maintenance of communications for feeding materials to the many isolated points at which work was in progress at any one time, and avoidance, if possible, of suspension of portions of the work for long periods when considerable areas of the river bed would be covered by water too shallow for the passage of floating craft and too deep and running too strongly for access by sand

embankment. In this respect, as has already been mentioned, the contractors were fortunate in the particularly dry river-bed conditions met with in the late working-season of 1933, which greatly simplified the arrangements for the virtual conclusion of the main well-sinking in that year, and put the completion of span-erection in the following working season beyond any reasonable doubt.

The Author, who was the Contractors' Agent throughout the undertaking, wishes to take this opportunity of expressing his very great appreciation of the skill and patience displayed by the foremen and all responsible members of the Contractors' staff, to whose unremitting efforts the successful fulfilment of the contracts is so very largely due. Mr. A. L. McIntyre, B.E., Assoc. M. Inst. C.E., was Sub-Agent and acted as Deputy Agent during the Author's absence from the site. Mr. C. R. Marshall was Senior Assistant Engineer. Mr. D. G. Anderson, B.Sc., Assoc. M. Inst. C.E., was in charge of the railway construction and Mr. G. C. Benson was in charge of compressed-air work and well-sinking generally.

The Author also wishes to convey his very sincere thanks to all those who have assisted him in the preparation of this Paper, and particularly to Mr. H. E. Whitehouse, M.C., M.A., Assoc. M. Inst. C.E., the chief assistant to the Resident Engineer, for his help and suggestions in connection with it. His thanks are due to the Cleveland Bridge and Engineering Company, Ltd., and to the joint Consulting Engineers, Messrs. Livesey and Henderson and Messrs. Rendel, Palmer and Tritton, for permission to present the Paper. The Consulting Engineers were represented at the site by Mr. F. W. A. Handman, C.B.E., M. Inst. C.E., as Resident Engineer.

The Paper is accompanied by seven sheets of drawings, from some of which Plate 1 and the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

Medical Notes on Work in Compressed Air.

For this work, out of seventy-one overseers, etc., and nine hundred and ninety-six natives examined, fifty-five and eight hundred and sixty-two respectively were selected. The age-limit adopted was 40 years. The medical examination covered the external habitus (rejecting skin-diseases, ruptures, varicose veins, ulcers, etc.), weight, height, thoracic perimeter, state of pulmonary, circulatory and renal organs, condition of nose and throat, and measurement of arterial tension. Special attention was paid to the time the pulse frequency required to revert to normal after physical exertion.

All natives working at the bridge had, as food-ration, 2¼ lbs. mealie-meal, 6 oz. ground nuts and ¾ oz. salt daily; 1 lb. fresh beef twice a week, and ¾ lb. dry fish or beans four times a week. To those working in compressed air the daily ration was composed of 3¼ lbs. mealie-meal, 1 lb. fresh beef, ¾ lb. ground nuts or beans, and ¾ oz. salt. They had also a dose of cod-liver oil (1 oz.) every day. To avoid danger from chills on leaving the decompression chamber, all natives were supplied with blankets.

Caisson Disease.

In Tables I, II and III cases are tabulated for convenience as medical air-lock treatments, hospital cases and deaths. Actually, all the cases had first been through the medical air-lock, and only when this treatment produced no effect were they taken to hospital. Medical air-lock cases in these Tables mean cases of "bends."

Table I.—Hospital Cases (Staff).

Diagnosis.	Pressure in lbs. per square inch.					Total.	Days off duty.
	Under 30.	30–35.	35–40.	40–45.	Over 45.		
Bends	—	—	5	5	1	11	33
Otalgy	2	—	—	2	—	4	19
Total	2	—	5	7	1	15	52

Table II.—Hospital Cases (Natives).

Diagnosis.	Pressure in lbs. per square inch.					Total.	Days off duty.
	Under 30.	30–35.	35–40.	40–45.	Over 45.		
Bends	—	1	—	7	1	9	73
Otalgy	1	—	3	3	—	7	28
Paraplegia	—	1	—	—	—	1	10
Labyrinth hæmorrhage .	—	—	1	—	—	1	7
Osteomyelitis	—	—	1	—	—	1	217
Metastasic abscesses . .	—	—	—	—	1	1	39
Total	1	2	5	10	2	20	374

TABLE III.—DEATHS (NATIVES).

Diagnosis.	Pressure in lbs. per square inch.			Total.	Days in hospital.
	35–40.	40–45.	Over 45.		
Pulmonary apoplexy .	1	—	—	1	1 (a few hours)
Cerebral congestion . .	1	—	—	1	2
Embolism	1	—	—	1	1 (a few hours)
Myelitis	—	1	—	1	6
Myocarditis	—	—	1	1	1
Total	3	1	1	5	11

Amongst hospital cases, "bends" still take the first place. These men, when medical air-lock treatments proved useless, were taken to hospital, where massage, hot fomentations and "mobilization" were applied.

"*Otalgy*."—This name was adopted for want of a better designation. The pain was always very acute the first day. No discharge or lesion of the tympan was noticeable. The treatment consisted of local baths with boracic lotion and dioxogen, and applications of carbolic glycerine. Single cases of paraplegia, labyrinth hæmorrhage, osteomyelitis and metastasic abscesses constitute the only note of some interest.

Paraplegia.—15th November, 1932. Native Joe. Well No. 30. Pressure 32 lbs. The man was taken to the medical lock in a subconscious state and with both inferior members paralysed. Conditions were not changed with this treatment and he was transferred to hospital. The main symptoms consisted of paraplegia and retention of urine. The idea of hematorachis at once occurred. Apart from a few catheterisms, the only treatment was adrenalin and pituitrin hypodermically. Next day his micturition was normal and one day later the paraplegia had disappeared.

Labyrinth Hæmorrhage.—23rd March, 1933. Native Santo. Well No. 31. Pressure 35 lbs. Symptoms: otalgy and giddiness. Standing, the man would turn round and fall if not helped at once. Here again pituitrin, combined with intestinal depletion and rest, seems to have been successful.

Osteomyelitis.—End of April, 1933. Native Dickson. Well No. 31. Pressure 39 lbs. According to his own history, this man felt, after decompression, an acute pain on the right leg below the knee. Afraid of hospitals and treatments, he did not complain and, with the help of some friends, made way to his home some 10 miles away from the work. The pains got worse and worse and the leg started to swell up. Sleepless, feverish, suffering unbearable pains, the man decided, after six weeks, to come to hospital, where he arrived on the 5th June, 1933. Temperatures ran from 100° to 103° F. The leg, especially its upper part, was very swollen. After palpation (he still persisted in concealing his history), a phlegmon was diagnosed. A large incision disclosed a few drops of pus. With drainage and adequate treatment conditions improved slightly, and for some time not much attention was paid to the case. Ulterior observation, after the full history was told, showed evident signs of osteomyelitis. Surgical intervention was decided on and after a large trepanation of the tibia, which gave way to a great quantity of pus, the general and local conditions of this patient began to improve, the first rather quickly but the latter very slowly. After 217 days he left hospital completely cured. Pre-

TABLE IV.—COMPRESSED-AIR WORK. CAISSON DISEASE: ANALYSIS OF CASES (EUROPEANS).

Pressures: lbs. per square inch.	Main Wells.							Extra Wells.								Totals.	Percentages.
		Well No.						Shifts.	Well No.								
		27	28	29	30	31	32		1A	2A	3A	4A	5A	6A	7A		
Under 30	Shifts worked (three of 8 hours)	—	—	—	—	—	446	Three of 8 hours	60	—	—	—	—	—	—	506	
	Medical-lock cases	—	—	—	—	—	2		—	—	—	—	—	—	—	2	0·4
	Hospital cases	—	—	—	—	—	2		—	—	—	—	—	—	—	2	
30 to 35	Shifts worked (three of 8 hours)	—	—	—	77	—	—	Three of 8 hours	126	108	42	—	—	42	—	395	
	Medical-lock cases	—	—	—	1	—	—		1	—	—	—	—	—	—	2	0·5
	Hospital cases	—	—	—	—	—	—		—	—	—	—	—	—	—	—	
35 to 40	Shifts worked (four of 6/4 hours)	—	—	—	120	618	—	Four of 6/4 hours	151	216	156	—	—	64	—	1,325	
	Medical-lock cases	—	—	—	1	32	—		—	—	—	—	—	—	—	33	2·5
	Hospital cases	—	—	—	1	4	—		—	—	—	—	—	—	—	5	
40 to 45	Shifts worked (six each of two 2 hours twice daily)	117	621	757	351	54	—	Six of 2 hours twice daily	—	62	132	156	504	163	36	2,953	
	Medical-lock cases	6	13	5	—	—	—		—	—	1	—	4	—	1	30	1·0
	Hospital cases	—	6			—	—		—	—	—	—	—	—	—	7	
Over 45	Shifts worked (six each of two 2 hours twice daily)	144	90	—	—	—	—	Six of 2 hours twice daily	—	—	—	120	84	96	300	834	
	Medical-lock cases	1	3	—	—	—	—		—	—	—	—	—	2	—	6	0·7
	Hospital cases	—	1	—	—	—	—		—	—	—	—	—	—	—	1	
Totals	Shifts worked	261	711	757	548	672	446		337	386	330	276	588	365	336	6,013	
	Medical-lock cases: Nos.	7	16	5	2	32	2		1	—	1	—	4	2	1	73	
	,, percentages	2·7	2·2	0·66	0·36	4·8	0·45		0·3	—	0·3	—	0·7	0·6	0·3	—	1·2 (1 in 5)
	Hospital cases	—	7	—	2	4	2		—	—	—	—	—	—	—	15	

TABLE V.—COMPRESSED-AIR WORK. CAISSON DISEASE: ANALYSIS OF CASES (NATIVES).

Pressures: lbs. per square inch.	Main Wells.	Main Wells. Well No. 27	28	29	30	31	32	Extra Wells. Shifts.	Extra Wells. Well No. 1A	2A	3A	4A	5A	6A	7A	Totals.	Percentages.
Under 30	Shifts worked (three of 8 hours)	—	—	—	—	—	8,548	Three of 8 hours	1,298	—	—	—	—	—	—	9,846	
	Medical-lock cases	—	—	—	—	—	22		3	—	—	—	—	—	—	25	0·255
	Hospital cases	—	—	—	—	—	1		—	—	—	—	—	—	—	1	
	Deaths	—	—	—	—	—	—		—	—	—	—	—	—	—	—	
30 to 35	Shifts worked (three of 8 hours)	—	—	—	1,478	—	—	Three of 8 hours	2,725	2,336	909	—	—	907	—	8,355	
	Medical-lock cases	—	—	—	80	—	—		9	5	—	—	—	—	—	74	0·88
	Hospital cases	—	—	—	2	—	—		—	—	—	—	—	—	—	2	
	Deaths	—	—	—	—	—	—		—	—	—	—	—	—	—	—	
35 to 40	Shifts worked (four of 6/4 hours)	—	—	—	2,289	11,828	—	Four of 6/4 hours	3,327	4,691	3,327	—	—	1,384	—	26,846	
	Medical-lock cases	—	—	—	61	129	—		4	3	7	—	—	—	—	204	0·76
	Hospital cases	—	—	—	2	3	—		—	—	—	—	—	—	—	5	
	Deaths	—	—	—	—	3	—		—	—	—	—	—	—	—	3	
40 to 45	Shifts worked (six each of two 2 hours twice daily)	4,464	23,690	28,496	13,390	1,030	—	Six of 2 hours twice daily	—	1,298	2,855	3,370	5,450	3,374	779	88,196	
	Medical-lock cases	41	53	79	34	—	—		—	—	18	7	31	—	4	267	0·30
	Hospital cases	1	7	1	1	—	—		—	—	—	—	—	—	—	10	
	Deaths	—	—	—	—	—	—		—	—	—	—	—	—	—	1	
Over 45	Shifts worked (six each of two 2 hours twice daily)	2,748	1,518	—	—	—	—	Six of 2 hours twice daily	—	—	—	2,595	1,817	2,076	6,488	17,242	
	Medical-lock cases	8	1	—	—	—	—		—	—	—	11	6	15	45	86	0·05
	Hospital cases	1	—	—	—	—	—		—	—	—	1	—	—	—	2	
	Deaths	—	—	—	—	—	—		—	—	—	—	—	—	—	1	
Totals	Shifts worked	7,212	25,208	28,496	17,157	12,858	8,548		7,350	8,325	7,091	5,965	7,267	7,741	7,267	150,485	
	Medical-lock cases: Nos.	49	54	79	155	129	22		16	8	25	18	37	15	49	656	
	,, percentages	0·70	0·20	0·28	0·90	1·00	0·25		0·20	0·10	0·35	0·30	0·50	0·20	0·70	—	0·43
	Hospital cases	2	7	1	5	3	1		—	—	—	1	—	—	—	20	0·015
	Deaths	1	—	1	—	3	—		—	—	—	—	—	—	—	5	0·003

sumably an embolism of a tibial branch must have been responsible for this case.

Metastasic Abscesses.—29th March, 1934. Native Gunselo. Well No. 4A. Pressure 40 lbs. This man also came to hospital a few days after his accident In his history he said that he felt slight pains all over his body and a great fatigue after decompression. At the time of his admission, six superficial but very big abscesses were already formed. Treated surgically, he left hospital after 39 days.

At well No. 31, with a pressure of 35 to 40 lbs., three deaths occurred on the 9th, 25th and 26th March, 1933.

Pulmonary Apoplexy and *Embolism* cases were taken to hospital in coma and only lasted a few hours. The first case had great dyspnoea, but no hæmoptysis. Revulsion, camphorated oil, strychnine, etc., were tried without effect.

Embolism was verified in post-mortem. Infarctus of all pulmonary artery ground was found.

Cerebral Congestion case was also in coma for two days and showed symptoms of a left hemiplegia with retention of urine.

Well No. 31 was founded in a very compact clay and the circulation of air was bad. The atmospheric conditions also were bad, especially at the time of the first death (maximum temperature 105° F., minimum 70° F.).

In spite of that, however, these three deaths are unexplainable.

Well No. 29, at a pressure of 40 to 45 lbs., on 31st March, 1933, caused one more death, by *Myelitis.* This case (native Layo) can be compared to the case already described of *Paraplegia.* It must have been a hæmatorachis, only the symptoms were unusually intense. The man was in coma and after three days meningism developed, with pain and stiffness on the back of the neck. A lumbar puncture gave pus, but meningococci were not found. Intra-venous injections of septicemine and intra-rachidian injections of trypaflavine had no effect.

The last death—due to *Myocarditis*—occurred at well No. 27 on the 18th November, 1933. Having been on leave at the time, the Author is unable to give any details.

Discussion.

Professor Inglis.

Professor C. E. Inglis said that he welcomed the opportunity offering his congratulations to the Authors on the excellence of the Papers, and also on the parts they had played in the successfu completion of an engineering undertaking of the first order magnitude.

The Papers were complementary one to the other; one present the somewhat detached point of view of the resident engineer directin the work, and the other presented the more mundane point of vie of the contractor who, in the course of carrying out the wor inevitably found himself at variance with some of the condition and regulations contained in the contract. The description of th work from two different aspects gave a picture which stood out strong relief. Both Papers gave considerable prominence to we sinking, where that dual representation was of great value. M Handman was perhaps a little over-optimistic in assuming that a his readers would know the exact significance of the term "sinkin effort." Professor Inglis himself had to confess that it had not bee clear to him until he reached p. 399, where Mr. Howorth had th kindness to define it. Only then did he realize the true meaning o the graphs in *Fig. 16* (p. 342) of Mr. Handman's Paper. When h had first seen those graphs he had hoped that they would give son information about sinking-resistances, and about their connectio with the nature of the material through which the wells were sunk By close examination, however, he found that, provided the sectio of the wells and the weight of the concrete were known, the grapl only gave the height of the well which was protruding above groun level at the various stages of sinking. Mr. Handman's remark on p. 342 regarding the average sinking-effort seemed of doubtfu validity, as the graph only showed that the final sinking effort wa 4·7 cwts., and did not give the average sinking-effort. The dete mination of skin-friction was a question of outstanding importance but, as Mr. Howorth pointed out on p. 400, there was unfortunatel only one case in which a definite measurement was obtainable. Tha was in the case of well No. 27, which appeared from the drawings t have been sunk to a depth of about 100 feet; at one stage of sinkin the cutting-edge of that well was entirely free, so that the well wa merely supported by skin-friction and air-pressure. By reducin the air-pressure until the well began to sink an accurate estimat

Professor Inglis.

could be obtained of skin-friction, the value found being $4\frac{1}{4}$ cwts. per square foot. It would have been valuable if many more observations of that nature could have been obtained. The conditions for making them were ready to hand, but a contractor could hardly be expected to spend time and money on such researches without adequate encouragement.

Mr. Handman had stated that no major difficulties in sinking wells had been encountered, but it was evident from the Paper that the technique of well-sinking had not yet reached finality. The great resistance offered to sinking in clay, even when the clay was dredged to a depth of 10 to 17 feet below the cutting-edge, was very remarkable, and Mr. Howorth's explanation that the failure of the wall of clay to break down under those circumstances could be attributed to the arch-action arising from the comparatively small bore of the cylindrical excavation seemed to be a reasonable one. However, human ingenuity should be able to devise some grabbing mechanism whereby the excavation could be carried almost to the limit of the cutting-edge; also, in sinking through granular materials the amount of kentledge required might perhaps be greatly reduced or even eliminated if a rapidly-vibrating load were applied so as to substitute dynamic friction for static friction, and to prevent the well from "going to sleep."

The question of impact-allowances was always of interest. For the Zambezi bridge an impact-load of 71 tons per bearing was mentioned; did that mean that the total impact-allowance on girders was going to be the very high figure of 284 tons? What allowance had been made for wind-pressure, and had high wind-velocities been taken into account? It was to be hoped that the allowance was of a more enlightened character than the easy method of allowing 50 lbs. per square foot when the bridge was unloaded, and reducing it 30 lbs. per square foot when a train was on the bridge. In that method it appeared to be assumed that when a high wind was blowing the trains ceased to run; but that was not perhaps quite so ridiculous as it sounded, for if the wind-pressure were 50 lbs. per square foot, a train would simply lie down on its side!

The choice of the span had presumably received careful consideration, and he would like to know how the figure of $262\frac{1}{2}$ feet had been determined. There was a well-known and logical foundation for the belief that in a multiple-span bridge the maximum economy was achieved when the cost of the superstructure approximated to the cost of the foundations and piers. In designing the Zambezi bridge, where the sinking of the piers was rather an unknown quantity, the engineers might have been expected to play for safety by putting

Professor Inglis.

rather more money into the superstructure and rather less into t piers. Actually, the excess seemed to be in the other direction, it was stated that the ratio of the cost of the superstructure to t cost of the piers was 0·92 for the main spans and 0·84 for the seconda spans. The obvious inference from those figures—though th inference might well be entirely wrong—was that the spans we rather too short for the maximum economy.

Mr. Codrington.

Mr. W. M. Codrington remarked that as Chairman of the Centr Africa Railway, which was responsible for the bridge, he ha been in close touch with the work throughout. He would n attempt to discuss any of the technical points which had been admirably set forth in the Papers, but he wished to draw attentio to the question of health, to which Mr. Howorth alluded. The si chosen for the Zambezi bridge was an ideal place for the breedin of mosquitoes, and he trembled to think what would have happene had not very careful precautions been taken to guard against tha danger. The advice given by the Ross Institute had been essential of a simple nature, but it had entailed a considerable degree discipline and care on the part of all those in the bridge zone, whe there had been a very heterogeneous temporary population of som thousands of Europeans and natives drawn from many differen territories. The discipline and organization needed to guard again the danger of disease would never have been achieved if the tw men particularly concerned—the Authors of the Papers—had no had very high personal qualities, and he desired to take the presen opportunity of paying his tribute to their success.

An unusual feature of the work was that the money for buildi the bridge had been advanced by the British Government to a priva commercial company. The company had naturally had to rely c its technical advisers, but had felt very safe in that respect, havi the support of two famous firms of engineers—Messrs. Rend Palmer and Tritton and Messrs. Livesey and Henderson. S Robert Gales had visited the site in the early days, and Mr. Codringto had accompanied Sir Brodie Henderson when he had visited t bridge just after construction had started. He wished to take t opportunity of paying a tribute to Sir Brodie Henderson, T Institution's very eminent Past-President. The Zambezi brid was the child of the later years of his professional life, and on it had lavished the fruits of a very varied experience gained in ma parts of the world. Sir Brodie was the ideal consulting engine and to those on the Board of the Company who were his clients had been much more than merely a technical expert. In him th had had a trusted counsellor and a wise friend to whom they h often appealed, and never in vain, for help and advice. The Zamb

bridge would perhaps be regarded as Sir Brodie's greatest achievement and, in that sense, his memorial. Mr. Codrington.

Mr. H. J. NICHOLS observed that the very great length of the bridge was notable, in view of the comparatively small discharge and maximum velocity of the river during high flood; that velocity was in fact so low that it had apparently been considered unnecessary to protect either abutment. Had the question of building training-works been considered? The effective length of the bridge might thus have been reduced to approximately the width of the wetted channel, or, at least, the viaduct and the secondary spans might have been eliminated, and the navigable channels might have been improved and perhaps stabilized. That method of approaching a bridge problem in such a river was almost standard practice in India. Mr. Nichols.

The span adopted seemed to be short in comparison with the size and depth of the piers; further, the spans were only designed for a single 3-foot 6-inch gauge track, so that the weight of steel per foot of girder would be quite small. It was stated on p. 367 that the ratios of the cost of the superstructure to that of the piers were 0·92 and 0·84 for the main and secondary spans respectively. With the help of the figures given in the Papers he had deduced a price of £42 per ton in place for the steelwork of the main spans, and £42 10*s*. per ton for that of the secondary spans. Those figures might not be exact, but they did appear to be unusually high, and were approximately double the usual price of steelwork erected in India. Difficulties of transport and other contingencies might have accounted for the relatively high price of the steel, but, in view of that price, it seemed strange that the viaduct at the end of the bridge, if it had to be built, had also been built of steel, instead of employing reinforced concrete. The cost of one of the major piers seemed to be about £15,000. That compared with a figure of about £14,000 for a somewhat similar pier sunk to a similar depth at the Nerbudda bridge in India, which carried two heavy tracks. As the pier costs were not dissimilar from those ruling in India, whereas the steel costs were about twice those in India, it was to be expected that the general profile of the Zambezi bridge would differ greatly from those of corresponding bridges recently erected in India.

Why had small horizontal sub-struts been adopted to support the main verticals of the main trusses? Those struts produced very unpleasant secondary stresses; the main verticals were not particularly long—about 37 feet between gussets—and it was possible that they could have done their job without support. The impact-loading allowed on the bridge seemed to be unusually high.

With regard to the use of 1 : 3 : 6 concrete for the bottom plugs,

Mr. Nichols. which had been placed under water, it seemed that some of that cemen would have been washed out during placing, and that the concret might therefore be considerably weaker than 1 : 3 : 6 when place He desired to add to the Authors' praises with regard to the exceller dome attachment for well-sinking, designed by Mr. Fereday. H himself had also had occasion to use it, and had found it to be of th greatest value. The foundation-loads applied were of interes The live load on the main piers was only 0·83 ton per squar foot, or 11·4 per cent. of the continuous dead load, and he woul like to ask whether, in such deeply-founded piers, it was necessar to take into account the live loading, provided it were within som agreed limit—perhaps 25 per cent. In testing so large a pil no one would accept a test if the test-load were left on for only a fe minutes ; and, after all, a live load was usually of equally short applica tion. The figure of 11·4 per cent. might be regarded as a measure efficiency of the piers in doing their job—which was merely to carry pay load—and it seemed to be rather a low one. Incidentally, if th live load were to be disregarded a pier would be designed solel from the point of view of convenience in sinking, and it did appea as Professor Inglis had suggested, that pier design had not ye reached finality. While the pier was being sunk its weight was th engineer's best friend, but as soon as the pier was founded and plugge at the bottom it became his worst enemy. A circular cross-sectio was certainly the easiest to deal with in sinking, and he desired to as whether the question of using a single 20-foot diameter well for eac of the Zambezi bridge piers had been considered. If that had bee done, the same diameter of dredging-well being adhered to, th sinking-effort would have been increased by 34 per cent. ; or i the sinking-effort had been reduced to its present value, then th final total live and dead load on the base of the pier would have bee increased only from 8·17 to 8·2 tons per square foot.

Dealing with the track, he questioned whether the sleeper-spacin of 2 feet with 10-inch-wide sleepers was in fact sufficiently clos to prevent a derailed wheel from going through. In India on metre gauge track the corresponding dimension was 16 inches with a 8-inch sleeper, leaving an 8-inch clear space. The anti-creep plate shown in *Fig. 25* were practically identical with the four-key ste sleeper used in India ; from experience there it had been found tha those four keys did not provide an absolute anchor, so that the rai did require some attention at fairly frequent intervals.

The practice of removing concrete test-specimens for speci curing was open to criticism. It would appear that in order to b representative of the concrete in a pier they should remain on th pier and thus be cured under the same conditions as the pier itsel

For how long had the concrete in the piers been kept wet before being allowed to dry? Mr. Nichols.

He agreed with the opinion expressed on p. 363 regarding the removal of mill-scale, but it was an operation of considerable difficulty. Something might be done to loosen the scale during the last few passes through the rolling-mills. It had been his experience that the scale on steel rolled in England was slightly easier to remove than the scale on steel rolled in India, and there might be something in the manipulation of those last few passes.

Mr. Howorth was to be congratulated on having carried through such a great work in $3\frac{1}{2}$ years. Those concerned had been fortunate in meeting with so much clean sand through which to sink, so that steady progress could be made throughout the major part of the work. From the progress-chart it would appear that seven piers had been sunk from floating sets, and that compressed-air sinking had been used in the case of five; when working from Pier No. 1, however, there appeared to be a considerable interval between the completion of the piers and the erection of the steelwork. Had that been due to the difficulty of arranging the arrival of the steelwork at the site just when wanted, whilst avoiding too great an accumulation of steelwork on ground which was subject to inundation?

It would appear that the difficulty in pitching pier No. 32 might have been overcome had the pier been water-borne and fitted with sea-cocks to enable the last few feet of sinking to have been done rapidly before any appreciable scour took place. That idea had been used in India, and had in all cases proved to be extremely effective.

It was interesting to read on p. 401 that some of the wells were inclined to "hang" when they were in sand a few inches above clay. Similar trouble had been met with in India, and on such occasions large lumps of sand conglomerate had been brought up. It appeared that there was a tendency for sand overlying clay to become consolidated in that manner, and it was possible that a similar cause of trouble had been present in the Zambezi well-sinking. Finally, he would like to question the practice of sinking piers to a depth at which they were considered to be immune from scour, and at the same time entering on the never-ending process of dumping rubble around them in order to prevent that scour. It would appear to be quite unnecessary to adopt both precautions.

Mr. Ernest Bateson remarked that the Papers gave clear evidence of the co-operation which had existed between the Authors during the construction period, and which had contributed largely to the successful completion of the work. Mr. Howorth referred to difficulties which had occasionally been encountered, but when account was taken of the magnitude of the work, the difficult climatic Mr. Bateson.

Mr. Bateson. conditions, the scarcity of skilled labour, the fact that the two ends of the bridge were in different territory, and the numerous authorities, companies and persons concerned, it would be agreed that those difficulties had been surprisingly few, and that the satisfactory progress of the work was evidence of the cordial co-operation of all concerned.

Mr. Howorth, on p. 398, referring to the sinking of wells Nos. 2[illegible] to 28 inclusive, stated that all the wells had given trouble, but examination of the well-sinking chart did not indicate that there had been any substantial reduction in the average rate of sinking. Well No. 24 had refused to sink by open dredging beyond 106 feet below low-water level, in spite of the addition of 1,000 tons of kentledge combined with 35 feet of pumping. That refusal had been due to the inability of the grabs to remove hard material more than 2 feet below the cutting edge. In that case the method of sinking by open dredging had ceased to be practicable, and further sinking would only have been possible by removal of the material either under compressed air or by other methods. The delay with wells Nos. 25 and 26 to which Mr. Howorth referred had been due to the rise of the river, and not to difficulties in sinking. On pp. 39[illegible] and 399 Mr. Howorth made several references to contract-depth. By that he understood him to mean the depths shown on the drawings issued to the contractor before the work had been commenced. The specified depths for wells founded in rock were as given by Mr. Handman on p. 340, whilst the depth specified for wells founded in sand was 110 feet below low-water level. It was not specified that 110 feet should be the maximum depth for any well, and it had not been intended that any well would be accepted for founding at that depth unless it was in acceptable material. The contract schedule had included an item for sinking by open dredging below 100 feet below low-water level, and did not limit sinking by that method to a depth of 110 feet. Wells Nos. 23, 25, and 26, when they had reached a depth of 110 feet, had been in very poor material, and the extra sinkages of 11 feet, 14 feet, and 4 feet respectively had been fully justified and had carried them into much better material—soft sandstone, in two cases. The maximum depth specified for the use of compressed air had been 100 feet below low-water level, and the contractor was to be commended for employing compressed air below the specified limit in the case of well No. 27, thereby enabling it to be firmly founded in soft sandstone rock.

The Papers dealt with the work as constructed, and a little supplementary information regarding the design of the bridge might therefore be of interest. On the contract drawings the bridge was shown as built, except that the portion now occupied by the seven secondary

Mr. Bateson.

spans had been originally shown as viaduct. The information available regarding the behaviour of the river-bed during the flood-season had been incomplete, and the resident engineer had been instructed to carry out further investigations during his first season at the site. From the first it had been recognized that the viaduct could only be regarded as a permanent bridge if the main channel of the river remained within that portion bridged by the thirty-three main spans. The investigations carried out by the resident engineer disclosed the presence of more or less active channels in the neighbourhood of the junction of the main spans and the viaduct, and it was considered advisable to add further spans carried on piers founded on wells, and to shorten the viaduct portion accordingly. In the meantime the driving of the piles for the viaduct had disclosed the presence of a hard stratum, which had been diagnosed from borings as sandy clay. Consideration of the question of adding further spans showed that, in consequence of the occurrence of that hard stratum at a relatively high level, it would be more economical to utilize spans of a shorter length than the main spans. As a result seven secondary spans of 165 feet each were substituted for an equivalent length of viaduct. Consideration of that important modification had been responsible for suspension of the pile-driving operations for a period of 8 weeks, and its adoption was responsible for the discontinuity referred to by Mr. Howorth on p. 405, as it prevented Mr. Howorth from connecting up the viaduct with the main piers already completed.

Although the hard stratum, which Mr. Howorth, for want of a better name, had described as fairly hard decomposed sandstone, was hard enough in situ to resist displacement by the pile-tube with its blunt-nosed shoe, pieces of it were easily crumbled between the fingers, and when placed in water had been found to disintegrate with surprising rapidity. The material was in fact nothing more than sand inefficiently held together by a natural binder, and it had been realized that if the channels previously referred to became active the overlying sand would be scoured away and the hard stratum would also be affected. It was obvious that the clauses of the specification regarding wells founded in rock were not applicable to that material, and the secondary wells had therefore been taken sufficiently deep to ensure security in the event of scour of the river-bed. The deepest penetration into that material had been about 26 feet, giving a total depth of 73 feet below low-water level, as against 110 feet in the case of main wells founded in sand.

The bridge was required to comply with the Portuguese Government's regulations, which included the making of tests under a train consisting of two of the heaviest locomotives in use on the line,

Mr. Bateson. together with a sufficient number of loaded wagons to cover th longest span. Stress-readings were to be taken in members selecte by the Government Engineer, and the deflexions of the various span were to be measured. Calculated stresses and deflexions had ha to be supplied to the Government Engineer, and the regulation specified that the actual stresses and deflexions should not excee them by more than a specified percentage. As it had not bee known which members would be selected for the tests, the calculate stresses in every member in each different type of span had had to b supplied. The Fereday-Palmer stress-recorder and the Fereda deflectometer had been used for carrying out the tests. As was usu in such cases, the recorded stresses and the deflexions had all bee slightly less than the theoretical figures. Mr. Handman and M Howorth having left the site, the tests had been carried out b Mr. Whitehouse, who had then joined the staff of the Railwa Company, Mr. Learmouth, who had been acting Resident Enginee and Mr. Anderson, who represented the contractor. Their cordia co-operation with the engineer representing the Portuguese Govern ment and with the Railway Company's staff had enabled the tests t be carried out most expeditiously and without hitch of any kind.

Hon. Philip Henderson. The Hon. PHILIP HENDERSON remarked that the question o shortening the bridge and of regulating the river-channel by mean of a bund had been fully studied, but it had been agreed that the us of a bund would have been extremely dangerous, as the river ha most peculiar habits; it might even have caused the river to chang its course. When the river was in normal flood it was probabl not more than 3 miles wide, but during a high flood the countr was inundated for perhaps 40 miles. In 1922 Mr. C. Seager Berry M. Inst. C.E., had been sent out to study the conditions governin the choice of the bridge-site. He had been followed by Mr. R. J Hallidy, who had had a very wide experience of Indian rivers, an it had been originally his idea that a bund should be employed but he had subsequently come to the conclusion that a bund wa absolutely impossible.

*** The Correspondence on the foregoing Papers, together with the Authors' replies, will be published in the Institution Journa for October, 1937.—SEC. INST. C.E.

ORDINARY MEETING.

8 December 1936.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President, in the Chair.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5106.

"The Maintenance of Waterways to Harbours and Docks."

By Raymond Carpmael, O.B.E., M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

Of the many problems with which the civil engineer has to deal, it is doubtful whether any give him greater concern than those directly connected with tidal waters. No other description of constructional work, although calling for the most skilled knowledge and for the fullest study of local conditions, can cause an equal measure of uncertainty and anxiety as that connected with works mainly affected by ever-changing tidal conditions. This could be amplified by indicating that the majority of the main engineering problems in connexion with harbours and docks are due in a greater or lesser measure to water-action, of which some instances are :—

(a) Alteration of tidal contours due either to the attrition of lands adjacent to the sea, with resultant destructive effect on structures constructed for the support of railway and other works, or to the accretion of materials carried in suspension by the sea, tending to reduce navigable depths at the entrances to harbours and docks.

(*b*) Destruction of structures and erosion of land caused by the increased weight and velocity of water passing down rivers in times of flood occasioned by heavy rainfall, or by rapid rises of temperature in districts where rivers have their source in mountainous country which is covered by snow or ice during winter months.

(*c*) Slips in railway and other embankments, to find a remedy for which the source of the influx or infiltration of water must be traced in the first instance in very nearly all, if not in all, cases.

The question of the maintenance of waterways to harbours and docks should appropriately be considered under two headings, namely, the causes of the accumulation of silt in navigable waterways and of the destruction of works constructed for their protection, and the methods employed to maintain these waterways.

Causes of Siltation.

The causes of the accumulations necessitating artificial works or operations to maintain navigable depths are (*a*) direct sea-action, causing coastal erosion in one area and deposit of the eroded material in another area, and (*b*) deposition of silt brought down from inland and carried, particularly during times of heavy rain, by rivers discharging into harbours and entrance-channels to docks.

All over the world unceasing changes are taking place in the seaboard or land-frontages to the sea. In some parts of the coast erosion of cliffs goes on, sometimes accompanied by denudation or stripping of the materials of which sea-beaches are composed, with the result that the sea encroaches on the land and the seaboard travels inwards. The converse of this occurs elsewhere, and by the deposition of materials from the sea, land is formed and the seaboard advances.

It has been conclusively proved that enormous quantities of eroded material are actually borne by the sea from place to place, such travel being known as littoral or shore drift. Deposits of such material from the sea on to the land are made wherever an obstruction, natural or artificial, is placed in the path of its free travel. One of the best examples of the former is Dungeness, projecting some 7 miles beyond the general line of the Kentish coast to the north-east of Hastings; its effect in stopping littoral drift is clearly illustrated by the fact that pebbles from Budleigh Salterton, near Exmouth, as well as Cornish pebbles, have been picked up there. Such pebbles are sea-borne, forming part of the great littoral drift which is mainly from west to east along the south coast of England

and of which the motive power is partly that of tidal currents and partly that of waves and currents generated by wind.

Between Hastings and Dungeness very large deposits of material are made, seriously affecting the ancient harbours of Rye and Winchelsea, whilst beyond Dungeness, that is to say, to the north-east, considerable erosion takes place. In 1925 there was advertised in the press the sale by public auction of some 1,000 acres of accreted land at Dungeness, which land had been recovered from the sea by purely natural agencies—the deposit of sea-borne material from the more westerly littorals of Devonshire and Cornwall.

Where such movements occur, any seaward projection from the coast line, whether natural or artificial, forms an intercepting barrier, and when such barriers, for example, breakwaters to provide sheltered waters in entrances to harbours, are constructed, the natural balance of quantities is disturbed. This disturbance may be helpful to places beyond the intercepting barrier where siltation due to accretion would otherwise occur, but it may also be detrimental to places where coastal contours can only be maintained by deposition of materials conveyed by sea action from elsewhere. The harbour engineer cannot evade this inexorable law, and wherever he creates a harbour in tidal waters, thereby causing interference with this littoral drift, he must sooner or later take steps to preserve the necessary depth of water for the safe navigation of the type of ship for which the harbour was designed.

Where natural harbours are situated at the mouths of rivers, a certain balance of quantities is reached between the reactions produced by the river and the tidal currents. The construction of harbour works at or near the confluence of a river and the sea must inevitably disturb this natural balance, and unless this is fully appreciated the subsequent maintenance-costs of such harbours may render their continued existence impracticable.

The Effect of the Construction of Artificial Barriers.

A few typical examples of the effect of interference with littoral forces caused by the construction of artificial barriers are shown in *Figs. 1* to *8*.* *Fig. 1* shows an unregulated river mouth. *Fig. 2* shows the same after regulation by two solid piers of equal length. *Fig. 3* shows the same after regulation by piers of unequal lengths. *Fig. 4* is similar to *Fig. 1* but with a projecting breakwater. This method has been adopted at Port Talbot, and also represents the condition of affairs at Swansea where the Gower Peninsula and Mumbles Head

* Based on diagrams on p. 223 of a Paper by Mr. A. E. Carey, "The Sanding-up of Tidal Harbours." Minutes of Proceedings Inst. C.E., vol. clvi (1903–4, Part II), *Figs. 12* to *15*, and *17* to *20*.

Fig. 1.

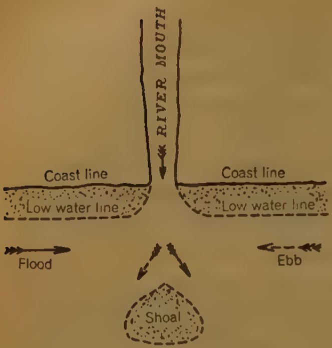


Fig. 2.

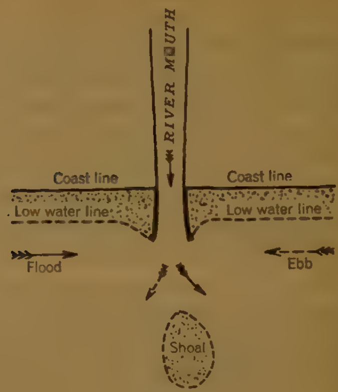


Fig. 3.

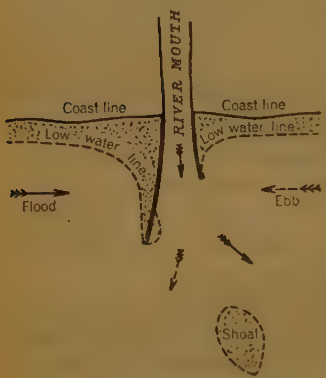


Fig. 4.

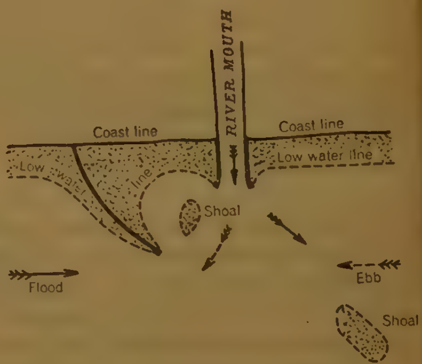


Fig. 5.

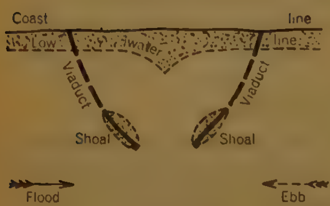


Fig. 6.

Fig. 7.

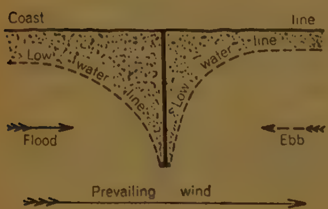
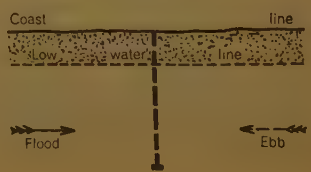


Fig. 8.



form a natural barrier corresponding to the north pier at Port Talbot. A shoal in a position corresponding to the one inside the breakwater in *Fig. 4* exists between the Mumbles and the harbour entrance, and is known as the "Green Grounds." *Figs. 5* and *6* illustrate different types of construction of (*a*) a solid and (*b*) an open breakwater projected from the foreshore. It will be appreciated that the direction of prevailing winds influences the contour of the low-water-line and also the position and shape of the shoals. *Fig. 7*, which shows the principle of the groyne for foreshore protection, indicates this effect. *Fig. 8* shows an open viaduct or pier which, unless designed with openings of sufficient span to suit local circumstances, may offer sufficient obstruction to cause shoaling.

Four actual examples may be cited of the effect of the interception of littoral drift consequent upon the construction of harbour works:—

(1) *Cearà harbour on the north-east coast of Brazil.*[1]—Here over £400,000 were spent in the construction of a breakwater nearly ½ mile in length, of which the first 250 yards consisted of open viaduct. The work occupied 10 years, but the harbour is now completely overwhelmed by sand with the exception of a small area where a shallow berth is maintained by dredging. Although a short length of open viaduct was provided, free sand travel was checked to such an extent by the solid portion of the breakwater that the whole harbour became choked.

(2) *Madras harbour.*[2]—Here two solid parallel breakwaters, 1,200 yards long, were projected from the foreshore, curving inwards at their extremities to form an entrance 500 feet wide, the position of which was subsequently moved and reconstructed to face approximately north instead of east. Siltation commenced even during construction of the work and has caused considerable trouble and expense ever since.[3] Unlike Cearà harbour, that at Madras has been maintained by dredging.

(3) *St. Catherine's, Jersey.*[4]—These works, consisting of a northern breakwater 2,000 feet long and a short southern arm, were commenced in 1848. The harbour is now sanded up and abandoned, with the total loss of the £200,000 spent on it.

(4) *The ancient port of Tyre at the eastern end of the Mediterranean Sea.*—The ancient city of Tyre, with the adjoining one of Sidon, was destroyed in 332 B.C. by Alexander the Great. The city, before that date, was partly on the mainland and partly on an island. The

---

[1] A. E. Carey, "Sanding-up of Tidal Harbours." Minutes of Proceedings Inst. C.E., vol. clvi (1903–4, Part II), p. 215.

[2] *Loc. cit.*

[3] Sir Francis Spring, "Coastal Sand-Travel near Madras Harbour." Minutes of Proceedings Inst. C.E., vol. cxciv (1912–13, Part IV), p. 153.

[4] *Loc. cit.*

former part was first captured and the materials from the demolished buildings used to build a pier or mole, bringing the island city within range of the *ballistae* and other engines of war from which large stones and other missiles were flung on to it. After the capture of the city the pier was extended, linking the island with the mainland. This interference with littoral drift caused the creation of a peninsula of a practically-uniform width of ½ mile and 1 mile in length, which was formed by materials carried by the sea and deposited on and around Alexander's narrow mole built for war purposes, a result which certainly was not anticipated by Alexander.

Formation of Bars at River Mouths.

A large proportion of the harbours of Great Britain have been built at the mouths of rivers which—and this is particularly the case with the Bristol Channel ports—bring down large quantities of alluvial matter eroded from the land. The deposition of this matter at or near mouths of rivers forms bars or shoals, accumulations which must be removed. These bars form obstructions to littoral drift, and collect, and are also built up with, material carried in suspension by the sea.

One of the best-known examples of these bars is that at the mouth of the Mersey, from which the enormous quantity of 9 million tons was dredged in 1934 by the Mersey Docks and Harbour Board to preserve open channels for the large liners using the port.

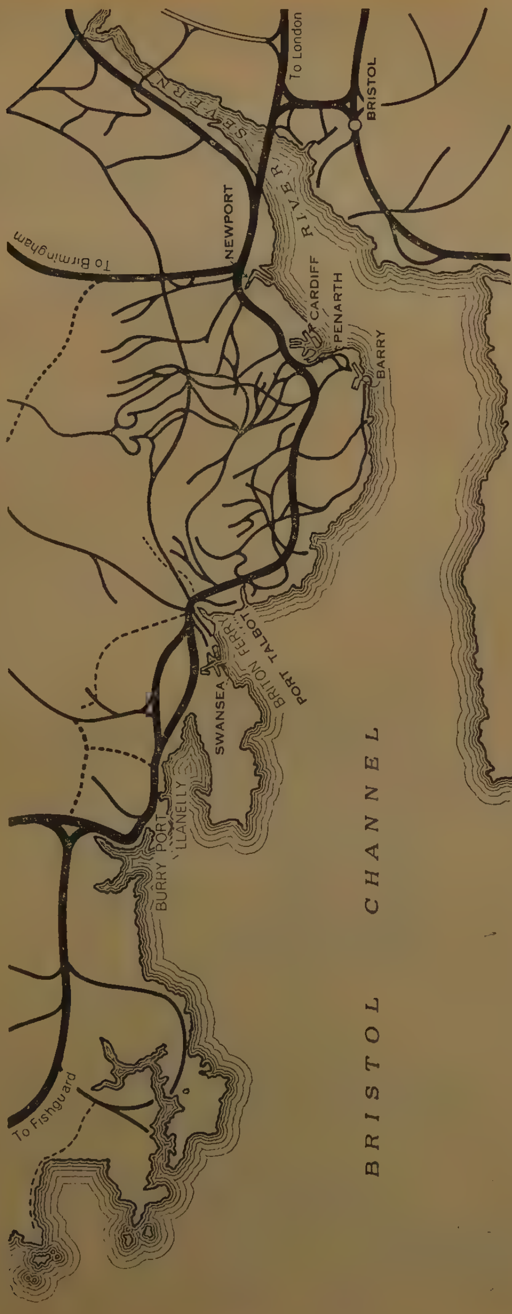
Great Western Railway Dock Problems.

This Paper deals mainly with the problems confronting the Great Western Railway Company in maintaining navigable depths in their Bristol Channel ports of Newport, Cardiff, Penarth, Barry, Briton Ferry, Port Talbot, and Swansea. At Fishguard, however, the circumstances are different, and little, if any, siltation is produced from sources outside the harbour.

Of the six main coal-exporting ports, three—Newport, Cardiff, and Penarth—are situated at the mouths of rivers, namely the rivers Usk, Taff, and Ely respectively, which flow into the estuary of the river Severn. These three rivers and the river Wye, which joins the river Severn at Chepstow, are highly charged with alluvial matter in suspension. The Severn itself is similarly heavily charged before reaching Chepstow. In addition to this alluvial matter the Usk and its tributary the Ebbw at Newport and the Taff and Ely at Cardiff bring down in times of flood very large quantities of coal-silt from the spoil tips of collieries situated in the valleys upstream.

Fig. 9 shows a general map, and diagrams of the principal South

Fig. 9.



Great Western Railway Ports in South Wales.

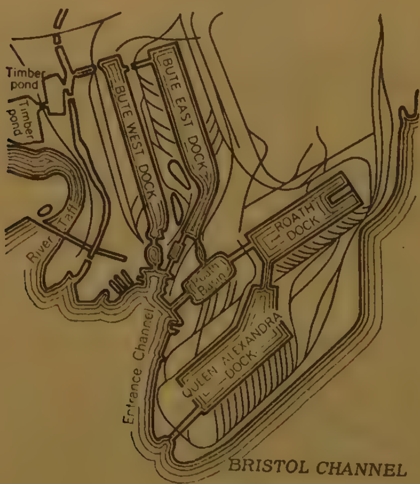
Wales ports are given in *Figs. 10–15* inclusive. Siltation at these ports from one cause or another may occur rapidly. At Barry,

Fig. 10.

NEWPORT.

where no river discharges and where both flood and ebb tides flow very strongly outside the entrance piers, siltation occurs less rapidly. At Port Talbot and Swansea the foreshores and sea-beds consist

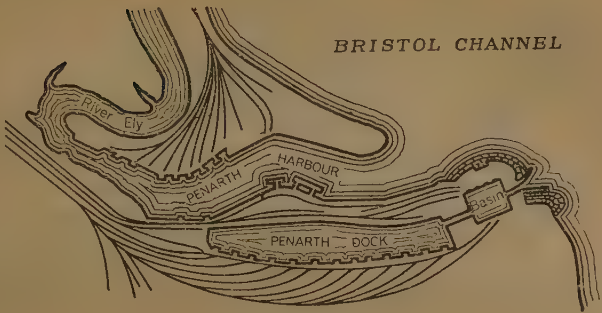
Fig. 11.



CARDIFF.

mainly of sand. The channels at both ports are subjected to siltation from sand, both wind-borne and carried by the sea. This siltation is carried into the docks (*a*) during operations of "levelling," that is to say, when the entrance lock-gates are opened at or about the

Fig. 12.



PENARTH.

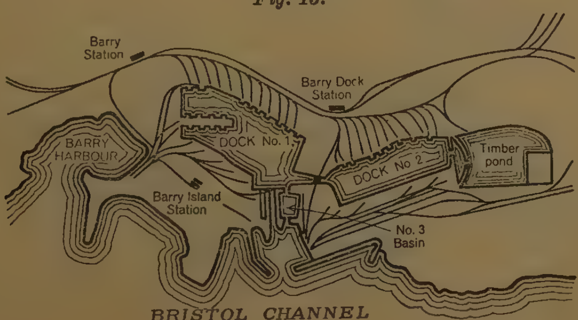
time of high-water of spring tides to obtain and impound the greatest possible depth of water in the dock. and (*b*) when the loss of water due to "locking" can only be compensated by pumping into the docks highly silt-charged water from the channels. Further, into certain of the docks there flow direct streams or feeders which, during their passage through colliery valleys among the Welsh mountains, are themselves often heavily charged with silt, sand, gravel, coal refuse, etc.

METHODS EMPLOYED TO MAINTAIN WATERWAYS.

The methods employed to maintain waterways can be considered under two headings :—

(*a*) Maintenance by artificial works, such as training walls, groynes, etc., to arrest and divert the travel of sand and shingle which would otherwise accumulate and reduce the depths.

Fig. 13.



BARRY.

Fig. 14.

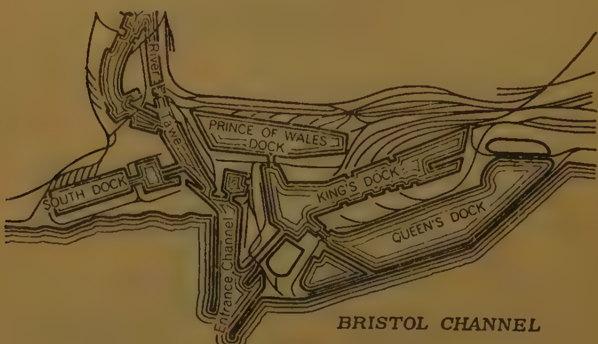
PORT TALBOT.

(*b*) Physical removal by dredging or sluicing of such accumulations due either to sea or river action.

There is an essential difference between training walls and breakwaters. The former are constructed at the mouths of rivers to control and accentuate their scouring effect. The latter are mainly, if not wholly, provided to form an adequate shelter. Both, however, projecting as they do beyond the coast line, arrest coastal drift, and, unless designed with a complete knowledge of circumstances, may occasion troubles far greater than the artificial maintenance of the necessary navigable depths.

In many harbours, entrance-piers are provided on either side of the approach-channels both for shelter and for navigation. Trouble often arises when such piers are of close timber construction, in that they are incapable of upholding the weight of accumulated coastal

Fig. 15.



SWANSEA.

drift. If openings are made to permit of the free passage of this drift material, it passes through and accumulates in the entrance-channel to the dock and must be removed by dredging or other means. To avoid such artificial removal, foreshore groynes may be constructed to arrest the drift before it reaches the harbour entrance. An example of this occurred at Penarth, where shingle-travel was arrested by a groyne, constructed at little expense, of old rails and sleepers. This aspect of the matter covers such a wide field that only one other example of protective constructional works will be given, namely the steps taken to check breaking-through of the river Taff into the main channel abreast of the entrance to the Queen Alexandra dock, Cardiff.

The river Taff follows here a sinuous course through tidal mud-flats and, as is generally the case with such rivers, tends to erode its concave banks and to build up by deposit the convex banks. At Cardiff the ultimate effect of this would have been the formation of a new mouth in the navigable channel abreast of the entrance to the Queen Alexandra dock, resulting in an increase there of siltation. To counteract this, brushwood groynes were built to arrest erosion, and at the same time the accreted spit in front of the opposite bank was dredged away to lead the river-discharge in a direction parallel to the main entrance-channel. These groynes comprised a line of 10-foot stakes spaced 8 to 10 feet apart and driven into the bed of the river. To these stakes, mattresses consisting of a light wooden framework interlaced with brushwood were fastened by light chains. The groynes varied in length from 60 to 100 feet and were placed at intervals of 150 to 200 feet along the river bank. After construction they were weighted by depositing on them heavy dredging-cargoes from steam hoppers. These works have proved successful, and their construction has relieved the Great Western Railway Company of the heavy additional dredging costs which the breaking-through of the river into the entrance channel would have involved.

In connexion with the best methods of maintaining waterways, the Author cannot do better than quote such an authority as the late Sir William Matthews, K.C.M.G., Past-President, who has put on record on p. 233 of the discussion on Mr. Carey's Paper [1] that "the whole art of conserving working-depths in and at the mouths of sand-threatened harbours lay in continual dredging." The Bristol Channel ports form no exception. Under favourable conditions, however, channels can, at least in part, be cleared by sluicing with impounded water released at or about the time of low tides, preferably the spring tides.

[1] *Loc. cit.*

Short Historical Survey of Dredging Craft.

Before describing the methods of removal of silt either by sluicing or dredging in harbours and docks, it will perhaps be of interest to make a short historical survey of such work, from which it will be seen that there is nothing new under the sun.

The progenitor of the bucket dredger was the Persian "chain of pots" used for raising water from wells. This consisted of an endless rope, or pair of ropes, made of palm fibre, to which were attached earthenware pots at uniform intervals. These pots, on passing over an upper terminal wheel 6 feet in diameter which was revolved by manual labour, discharged their contents into a trough through which the water was led for irrigation or other purposes.

The historian Pliny mentions the introduction of a similar pump into Greece either earlier than or coeval with the foundation of the city of Babylon. It is recorded that the working of these pumps by the daughters of the household was regarded as a very suitable form of punishment for their misdeeds.

The biblical reference in Ecclesiastes, chapter XII, in the words "or ever the silver cord be loosed or the golden bowl be broken at the fountain, or the wheel broken at the cistern," is evidently to this Persian "chain of pots." The silver cord is the palm fibre cord bleached white by the sun and glistening with minute globules of attached water; the golden bowl, the earthenware pots, golden in colour; and the wheel, the top terminal wheel, the breaking of which or the loosening of the cord causing it to slip off the wheel, would result in the dropping to the bottom of the well of the whole chain of pots.

A similar device was used for irrigating the "hanging gardens" of Semiramis, in Persia, where water was lifted through a height of 300 feet. It is still used under the name of *Sakia* in Egypt, a notable example being the raising of water from Joseph's well in Cairo, 156 feet deep. Modifications of this chain of pots, still operated by man-power but with the introduction of gear-wheels to give increased efficiency, were mentioned by historians between 1550 and 1600, and were used for the drainage of mines and for dealing with excavated solid materials. In Great Britain, prior to 1660, such machines were constructed with wood or leather barrels attached to endless long-link iron chains, and were known as "bucket gins" or "scotch gins." A modification of the chain of pots was the Chinese "chain" pump mentioned in Chinese literature in A.D. 1145, in which the earthenware pots were replaced by flat square pieces of wood.

These various devices were undoubtedly the progenitors, not only of the bucket dredger, but of all land conveyors and elevators, and the earliest adaptations of them were made by the Dutch for the removal, *circa* 1685, of silt from harbours and docks. A dredger, based on the Dutch "Chinese" pump model, was in use at Hull docks in 1785.

In the year 1750, during the demolition of Blackfriars bridge, water was raised from the foundation caissons by means of Chinese pumps. It may be of interest to know that a very modern and efficient French pump is based on this model, but consists of an endless spiral rotated at speed, the spiral performing the dual function of chain and buckets.

The first steam ladder-dredger used in this country was Sir Samuel Bentham's model made in 1800. The primitive *Shadoof* used in Egypt for many years, and still used for lifting water in buckets from the river Nile, has developed into the steam navvy and the dipper dredger which, in its most advanced form, is in use on the Panama Canal for dealing with land-slides in the Culebra Cut, where enormous quantities of debris and rock boulders up to 50 tons in weight are stated to have been removed by the buckets of the "dipper."

The *Shadoof*, a modification of which was largely used during the War for timber-loading, consists of an upright fixed pole carrying a horizontal arm capable of vertical and horizontal pivoting-movements. One end of the movable arm carries the bucket, the other end being elevated and depressed by man-power to fill and lift the bucket. Such machines were used in early days by the Dutch and Italians in dredging canals. An early adaptation in Great Britain was John Grimshaw's "spoon" dredger, built in 1796, and operated by a Boulton and Watt bell-crank engine of 4 HP. Dredgers of this type were used in Portsmouth dockyard between the years 1802 and 1806. Grimshaw's machine lifted 2 tons of soil per minute at a cost of from 1*d*. to 2*d*. per ton.

What may be described as the introduction of sluicing was carried out by the Venetians in the 14th century for the removal of bars or deposits of silt in harbours, when on ebb tides, these were stirred and scraped with iron harrows attached to long wooden handles which were operated by men in small boats. The ebb tide then carried the loosened sand into deep water. This method was also employed many years ago at Great Grimsby and on the Humber. A somewhat similar arrangement is in use to-day in the Great Western Railway Company's dock at Bridgwater. Here an iron drag or scoop at the end of a long pole is attached to the stern of a vessel by which it is dragged towards the mouths of sluices fixed

in the dock bottom. Shortly before low water these sluices are opened, and water from the dock rushes through the sluices and a system of culverts under the dock bottom, carrying with it the accumulated mud into the river, whence it is borne away by the ebbing tide.

To sluice an entrance-channel to a dock, sluice-gates, provided in the walls of the lock or in the lock-gates themselves, are opened at or slightly before the time of low water. The outpouring mass of water sluices or swills any accumulated silted matter away from the dock entrance. The sluices in the gates give a more direct flow of water than those in the walls, where the discharges on either side of the lock may interfere with each other, and so reduce the force of the flow of water. In few, if in any, cases does sluicing carry silt sufficiently far for it to be carried away by the main tidal currents, but it does at least prevent accumulation at the dock-entrance and to this extent reduces removal costs, in that it conveys the silt to a site whence it can be removed by bucket dredgers which cannot work in the vicinity of dock-gates owing to the restricted area of operation. Sluicing is, however, comparatively useless unless the depths of water over the dock sills at low water and the distance between the banks of the entrance channel are small. Thus no effective sluicing can be carried out at either Barry deep-water lock or Swansea, where sill cover never falls below 10 feet; but at the other South Wales ports where such depths vary from 5 feet to nil, sluicing can be effectively carried out.

At Port Talbot, with a navigable channel width of 180 feet and where strong west and south-west winds tend to pile up an accumulation of sand in the outer reaches of the channel, sluicing has proved of great value. Here, at a suitable tide, two steam hoppers are grounded in such a manner as to reduce the width immediately above the accumulation, that is to say, nearer the lock entrance. At low water the impounded dock water, with a head of some 20 feet, is released through the gate sluices and, after passing through the 40-foot channel between the hoppers, removes the accumulation of silt seawards. This operation is repeated over successive tides and is of particular value at Port Talbot where the range of swell is often too great for a bucket dredger to hold to its moorings.

It should be pointed out that the operation of sluicing, owing to the rapid discharge within a short period of time of large volumes of water, may occasion extensive erosion of the bed of the channel immediately in front of the dock-entrance apron, and thus undermine and otherwise damage the latter. To reduce this erosive action the apron should be so designed that the discharge of sluicing water at the lip should take place at an angle inclined slightly above the horizontal.

The lowering of dock water-level due to the discharge of sluicing water must be balanced by pumping in channel-water. Should this be highly charged with solid matter, as is often the case, additional dredging-costs are incurred in the removal of this solid matter after settlement within the dock-gates.

At Newport the waters of the river Usk carry in suspension approximately 35 cubic yards of solid matter per 1,000,000 gallons. Such highly-charged water occasions heavy maintenance-expenditure on the wearing portions of the impounding pumps and, if economically possible, its use should be avoided by the provision of an alternative feeder-supply from which, when necessary, excess of solid matter has been removed by deposition before reaching the pumps.

Types of Dredging Craft.

Modern dredgers are of four types: (1) sand-pumps and drag-suctions; (2) bucket or ladder; (3) dipper or spoon; and (4) grab.

The main difference between the types is that types (1) and (2) are continuous in action, either by the suction action of pumps or by elevation in buckets on an endless driver belt, whilst types (3) and (4) are intermittent.

Sand-Pumps and Drag-Suction Dredgers.

A typical example of the sand-pump is the one owned by the Mersey Docks and Harbour Board, which is capable of dealing with some 120,000 cubic yards a day of 10 hours, working in a depth of 70 feet. The drag-suction dredger is a development of the sand-pump and carries a special nozzle provided with rotary cutters and hydraulic jets for dealing with hard clay and soft rock. These dredgers have the advantage over the sand-pump type in that the dredged material is delivered into the hoppers with a minimum admixture of water. Typical examples of this type of dredger are those used in connexion with the construction of the Buenos Aires harbour, each capable of dealing with 10,000 cubic yards of clay in 12 hours. Many of the dredgers of these two types are provided with pumps to enable them to discharge the dredged material ashore for reclamation purposes.

No records exist of extensive trials of sand-pumps in the Great Western Company's Channel ports other than at Port Talbot, where the results were not altogether satisfactory. On p. 105 of a Paper by Mr. William Cleaver [1] a description is given of a 20-inch sand-

[1] "Alterations and Improvements of the Port Talbot Docks and Railways during the last Decade." Minutes of Proceedings Inst. C.E., vol. cxci (1912–13, Part I), p. 103.

suction plant, fitted with a specially-shaped nozzle and operated from the existing bucket dredger. This worked efficiently for some time, but after the adoption of the expedient of grounding craft to guide and increase the sluicing current, the channel is now almost entirely maintained by sluicing, and the use of the suction plant was discontinued.

The extension by Mr. William Cleaver of a short timber training-pier on the north side of the entrance channel by a tipped embankment of copper slag had an immediate and marked effect in reducing the rate of deposition of silt. Previous to this, silting often occurred to the extent of 5 to 6 feet in a few days, whereas later the same depth of deposit was hardly found in as many months. The embankment also served to check the travel of wind-blown sand from the adjoining beach.

At a later date, a sand-pump brought from Fishguard was fitted with a rotary cutter and a nozzle similarly shaped to that used by Mr. Cleaver, but for the reason given above was not kept long at work. With this exception no records exist of extensive trials of sand-pumps in the Great Western Company's Channel ports. Such trials, including those with the more efficient drag-suction dredgers, were not carried out as it was not anticipated that they would result in any improvement in economic working.

Bucket and Ladder Dredgers.

Dredgers which excavate and elevate material by means of an endless belt provided with steel buckets may be sub-divided into four classes :—

(*a*) Self-filling and self-propelled.
(*b*) Non-self-filling and self-propelled.
(*c*) Non-self-filling but dumb (incapable of self-propulsion).
(*d*) Self-filling but dumb (incapable of self-propulsion).

The selection of one or other of these classes, where bucket dredgers are used, must depend on local circumstances. In very narrow channels, where the presence of hopper barges alongside the dredgers would be detrimental to navigation, the self-contained carrier dredger is the only possible craft. It cannot, however, be described as economical, as during the whole of the time the craft is proceeding to and from the depositing-grounds, its main lifting engines are idle, so that its use should, if possible, be avoided.

Of the non-self-filling type, the provision of propelling engines is governed by how much shelter from bad weather exists in the channels to be dredged, and also by traffic conditions. Under the conditions prevailing in the Bristol Channel ports five bucket dredgers

are used in conjunction with steam hopper-barges. Of these dredgers two are self-propelling and three non-self-propelling, one of the latter being a vessel used by the former Cardiff Railway Company.

On purely economic grounds the case for the dumb dredger is strong, the additional running and overhead costs consequent upon the provision of an independent set of engines for propulsion amounting to some 15 per cent. This can, however, be reduced to about 10 per cent. by the provision of a belt drive to the top tumbler of the dredging ladder, which, as well as the propellers, can then be operated by one set of engines. Dumb dredgers have the disadvantage of their dependence on other craft for mobility, but no general principles can be laid down to assist in assessing the practical value of this mobility beyond indicating that, in channels of sufficient width to permit of the use of barges but where interference with traffic must be reduced to a minimum, and particularly where in times of storm dredging craft must move quickly to sheltered ground, advantage will be gained by the use of self-propelled craft.

Whether or not a bucket dredger should be provided with a hopper is largely a matter of opinion, but a hopper is certainly useful in collecting droppings from the buckets. In 1931, the "David Davies," one of the Great Western Company's dredgers, collected and deposited 5,400 cubic yards, the average for the 5 years 1930–34 being 4,000 cubic yards. The space occupied by a hopper would otherwise serve no useful purpose, but its provision results in a higher freeboard and so a longer working time for the attendant hopper barges.

Dipper or Spoon Dredgers.

Dipper dredgers follow the familiar lines of the steam navvy, and need no further explanation. They are largely used in excavating metalliferous deposits from the beds of rivers in Burma and elsewhere. The greatest development of this type of dredger took place for dealing with the enormous "slides" which occurred in the Culebra Cut soon after the opening of the Panama Canal. The dredgers were provided with 10- and 16-cubic-yard dipper buckets capable not only of lifting lumps of rock some 50 tons in weight but of actually digging to a depth of about 50 feet in soft unblasted rock—shale and sandstone—at the rate of from 7,000 to 10,000 tons per 24 hours, according to the hardness of the material encountered. The yearly output on the Canal of three of these craft was approximately 10,000,000 cubic yards.

Grab Dredgers.

Grab dredgers, mounting one or more cranes, are indispensable for use in docks for dredging coal-tip berths alongside quay-walls

and in other situations where a bucket dredger cannot conveniently work, and are largely so used in the Great Western Railway Company's ports. To reduce interference with dock traffic to a minimum, craft for this purpose should be self-propelled carriers, supplemented, where necessary, by large-capacity steam hoppers to reduce time lost in locking when proceeding to and from the depositing-grounds.

Although craft of this type more commonly deal with comparatively small quantities of material, an extremely useful general-purpose type is one mounting four grabs and of a hopper-capacity of 1,000 cubic yards. Grab dredgers so arranged have given excellent results on many large docks, and their use might with advantage be developed. The "Finn Mac Cool" operating at Buffalo, U.S.A., may be instanced as a typical example of a grab of large bucket-capacity. This craft has a 10-cubic-yard bucket operated by pneumatic power, with an output of 600 cubic yards per hour working in a depth of 65 feet.

Docks Organization (Engineering).

The Great Western Railway Company's South Wales ports are divided into two groups—the eastern: Newport, Cardiff, Penarth, and Barry, and the western: Port Talbot and Swansea. At Barry and Swansea Docks a Divisional Engineer is stationed responsible for all the civil engineering work of the harbours in these respective groups other than dredging, which is in the charge of a Dredging Assistant to the Chief Engineer, with a staff of Dredging Inspectors.

The Great Western Company also owns the harbours and docks at Briton Ferry, Burry Port, Llanelly, Bridgwater, Plymouth and Fishguard. These are in the charge of the Railway Divisional Engineers, and dredging is carried out at them by units detached from the main fleet and controlled by the Dredging Assistant. At Brentford and Chelsea, where the conditions are special, seasonal dredging work is carried out by contract. Dredging at the western ports is mainly carried out during the summer months owing to weather conditions, but at the eastern ports there is little interruption from this cause.

Statistics of Dock- and Channel-Dredging.

To indicate the extent of dredging essential to maintain the navigable waterways of the Great Western Railway Company, the largest private dock-owning corporation in the world, the following

statistics of dock- and channel-dredging, covering all their ports, are given :—

	Year 1935: tons.	Yearly average 1926—1935: tons.
Docks	769,542	1,038,617
Channels	4,661,013	4,111,703
	5,430,555	5,150,320

This represents an expenditure of a capital sum for dredging plant of nearly £500,000, with an average yearly cost for the 10 years ended 1935 of some £60,000 for operation, including ordinary running repairs. Approximately £45,000 of this sum represents the corresponding costs in respect of the entrance-channels to the South Wales ports, with which this Paper is mainly concerned. The ordinary maintenance-costs, that is to say, repairs up to £100, account for some 25 per cent. of this figure taking the average for the years 1933, 1934, and 1935. The cost of heavy repairs during those years averaged approximately 17 per cent. of the working costs.

Methods of Working Employed.

Loading of Hoppers.

Particularly in the eastern ports, the state of fineness of the material to be dredged and the ease with which it mixes with water render it impossible to fill the hoppers to capacity with solid matter. Should this be attempted, the surplus water passes over the side heavily charged with silt. At Newport, 1 cubic yard of silt weighs about 1·2 ton and, in general, at the other ports 1·25 ton. In the entrance-channel at Swansea, where the silt also consists of wind-blown sand, the equivalent is 1·30 ton, and at Port Talbot 1·40 ton. At the two latter ports the materials dredged for extensions of deep water in dock approximated to 1·80 ton per cubic yard and comprised sand at Swansea, and gravel, boulders and clay at Port Talbot. At Swansea this represents an equivalent of 1 cubic yard of virgin sand to 1·4 cubic yard of silt.

It is the practice when hoppers are loaded with silt and water to gauge the depth of the top of the mud below the well-coamings. This is ascertained by means of a graduated gauging rod provided with a circular foot 14 inches in diameter, and readings are taken at several points. These measurements are entered in the daily log sheet, and, by reference to Tables showing the capacity obtained by previous measurement of each hopper when loaded to varying depths below the coamings, the actual cargoes carried are easily ascertained.

Soundings.

Soundings are carried out systematically in the channels every 3 months, with occasional spot checks as found necessary, and, dependent upon the volume of shipping, this is also done in the docks. Wherever possible, soundings are taken by means of a steel wire 800 to 1,000 feet long with ferrules spaced 10 feet apart; fore- and back-sights are provided on shore and check-sights are taken from time to time on known shore marks, buildings, etc. The sounding chain is provided with a zinc sinker 7 inches in diameter and weighing 9½ lbs.

In the outer channels, soundings are taken as regularly as possible about every 12 feet along transits fixed by sextant readings as shore marks. Time readings are taken every 5 minutes with synchronized watches by one of the sounding-boat's crew and by a man stationed near the appropriate tide-gauge which is also read at these intervals. Copies of the sounding charts are supplied to the Chief Docks Manager for the use of his Dock Managers and Masters.

Depositing-Grounds.

The various ports are favourably situated in relation to depositing-grounds. The distances traversed by the carriers out and home when working in entrance channels vary from 3 to 8 miles according to the set of the tides and the positions in which the dredgers are working. At Penarth, Cardiff and Barry the maximum run does not exceed 4 miles. At Brentford and Chelsea the position is exceptional, as the whole of the dredged spoil has to be conveyed down the river Thames for a distance of 30½ miles before being deposited on the Essex marshes near Rainham.

Dredging-Costs.

The main factors influencing dredging-costs are :—

(*a*) an appropriate selection, distribution and economic use of the craft.

(*b*) losses of time due to weather.

(*c*) the exigencies of traffic.

Modernization and Centralization of the Great Western Railway Fleet of Dredgers.

Prior to the grouping of the railways in 1922, independent dredging units were maintained by the various South Wales railway companies now incorporated in the Great Western Railway Company. The fleet, which has been modernized in recent years and, as shown in Table I, includes several powerful craft well supplied with steam

hopper-barges of large carrying capacity, is now employed as one unit drawn for service at any of the Company's ports.

TABLE I.—SOUTH WALES PORTS—DREDGING CRAFT IN 1936.

Name and type of craft.	Capacity.		Dredging depths: feet.		
	Dredging per hour: average cubic yards.	Carrying: cubic yards.	Maximum.	Minimum: normal.	Minimum: abnormal.
Bucket self-propelling :—					
Don Federico	650	—	40	19	16
David Davies	750	800	50	22	18
Bucket non-self-propelling :—					
Peeress	800	—	45′ 11″	17	14
Foremost 49	1,000	—	45 and 60	18	16
Foremost IV	900	—	60	18	16
Grab dredgers :—					
Mudeford	50	287	—	—	—
Francis Gilbertson	100	305	—	—	—
Grabber	20	—	—	—	—
Graball	128	392	—	—	—
Steam hopper barges :—					
Viscount Churchill	—	890	—	—	—
Sir Ernest Palmer	—	890	—	—	—
Sir Henry Mather Jackson	—	890	—	—	—
Foremost VI	—	720	—	—	—
Foremost VII	—	720	—	—	—
Foremost 27	—	627	—	—	—
Foremost 44	—	883	—	—	—
Foremost 45	—	883	—	—	—
G.W.R. No. 1	—	925	—	—	—
G.W.R. No. 2	—	925	—	—	—

The tidal range in the Bristol Channel is very considerable, varying from 40·00 feet at ordinary spring tides at Newport to 26·80 feet at Swansea, with maximum recorded ranges at those ports of 45·30 feet and 32·40 feet respectively. This high range necessitates the employment of dredgers capable of dredging to considerable depths and with a wide range of ladder adjustment. Under such circumstances bucket dredgers, not capable of dredging below 45 feet, are limited in their economic usefulness.

Carriers.

An efficient hopper service is all-important, and the extended use by the Great Western Railway Company of steam hopper-barges

carrying upwards of 900 cubic yards, as compared with the older types, both steam and dumb, of capacities ranging from 500 to as low as 200 cubic yards, has very materially reduced the losses of time under the headings of "changing barges" and "proceeding to and from depositing grounds."

The carrier fleet at present consists of ten steam hopper-barges of a total capacity of 8,353 cubic yards, which have replaced thirty-one craft (twelve steam and nineteen dumb) with a total capacity of 8,900 cubic yards. This fleet operates with five bucket dredgers, a reduction of four on the number previously employed.

Effective Working Time.

Records kept for many years show that the average percentage increase in actual effective dredging time, that is to say, time occupied in lifting spoil, during the last 12 years, varies from 4 per cent. at Swansea to 16 per cent. at Cardiff where the effective time percentage in 1934 stood at 67·7 per cent. of the operating time.

A typical present-day record under the various headings under which the timing of dredging operations is divided shows that in 1933 bucket dredgers have been working 50 per cent. of the year, grab dredgers 84 per cent. and the steam hoppers 66 per cent., with a maximum for two of the barges of 88 per cent. Records are kept for each unit, showing separately for docks and channels the number of hours worked under the following headings:—starting and finishing, loading or raising, depositing and steaming, locking, traffic, weather, coaling, moorings, changing barges, awaiting barges, repairs, miscellaneous, and total. Average figures for different classes of craft are shown in Appendix I.

The periods in commission (periods during which actual loading was going on) vary from 58·6 per cent. to 66·9 per cent. of the overall time in the case of the bucket dredgers.

The following statement shows the employment of dredging craft during 1935:—

Type of vessel.	Total no. of days occupied in:			Average percentage of total time:		
	Working.	Repairs.	Lying-up.	Working.	Repairs.	Lying-up.
Bucket dredgers	926	267	706	48·8	14·0	37·2
Grab dredgers	748	68	125	79·5	7·2	13·3
Steam hoppers	1,967	319	846	62·8	10·2	27·0

Unit Costs.

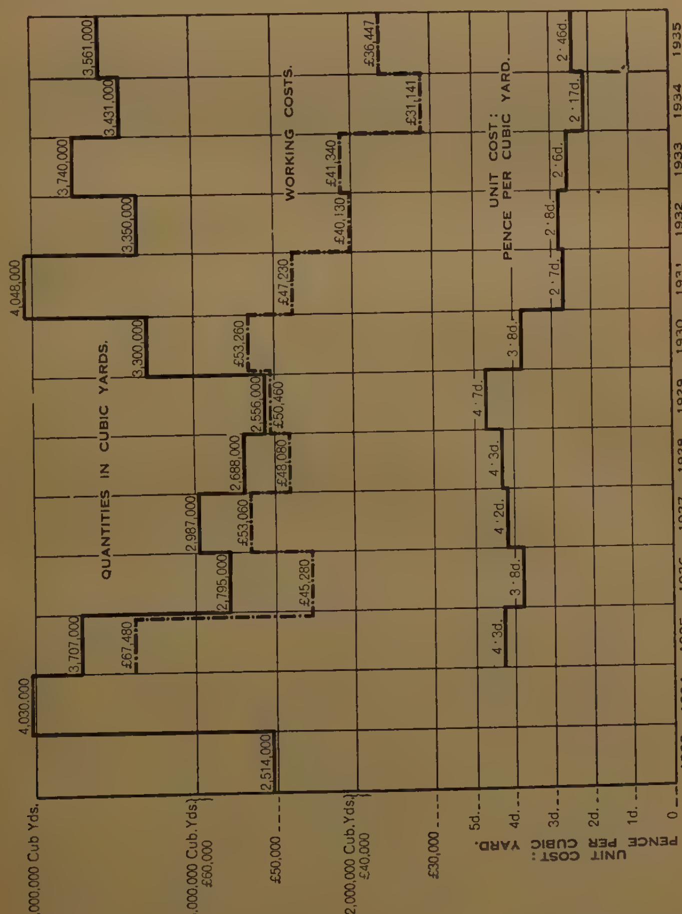
The graphic diagrams (*Fig. 16*) show the total and the working costs of channel-dredging at the South Wales ports over a period of

years, with actual quantities dredged. These figures are exclusive of periodical overhauls for heavy repairs and of depreciation charges.

Grab-Dredging Costs.

Dredging by grabs is carried out in ships' berths in dock, in entrance-basins, etc., and no extended use of such craft is made in channels.

Fig. 16.



COST OF CHANNEL-DREDGING AND QUANTITIES DREDGED.

Owing to the varied conditions under which these craft work, costs are very variable and would be misleading without knowledge of the particular circumstances.

Staff Arrangements.

The men employed with the Great Western Railway dredging fleet, including all units at the various ports, vary in number according to seasonal requirements from 190 to 80, of whom a nucleus is regularly employed, the remainder being engaged on a temporary basis as found necessary. The staff work under National Agreement conditions, the working hours being regulated to suit the depths of water available for dredging the tidal channels. Sleeping accommodation is provided on the new craft, the use of which is optional for the men.

The Paper is accompanied by four sheets of drawings, from some of which the Figures in the text have been prepared, and by three Appendixes, the first of which is reproduced as Table I, the second summarized in Appendix I, and the third summarized on p. 444.

APPENDIX I.

Analysis of Average Time Worked by Different Types of Dredging Craft.

Craft.		Starting and finishing.	Loading or raising.	Depositing and steaming.	Locking.	Traffic.	Weather.	Coaling.	Moorings.	Changing barges.	Awaiting barges.	Repairs.	Miscellaneous.	Total.
Five bucket dredgers, working in docks and channels.	Hours.	44·0	835·0	32·2	12·8	59·4	32·0	5·0	175·4	92·4	29·0	25·0	14·0	1356·2
	Per cent.	3·2	61·6	2·4	0·9	4·4	2·4	0·4	12·9	6·8	2·1	1·9	1·0	100·0
Two bucket dredgers, working in channels only.	Hours.	42·5	605·0	35·0	18·0	48·5	74·5	12·5	83·0	92·5	6·0	21·5	2·0	1041·0
	Per cent.	4·1	58·1	3·3	1·7	4·7	7·2	1·2	8·0	8·9	0·6	2·0	0·2	100·0
Three grab dredgers, working in docks and channels.	Hours.	47·7	1435·3	528·3	137·0	92·0	44·0	36·3	132·3	4·3	0	63·7	127·3	2648·2
	Per cent.	1·8	54·2	19·9	5·2	3·5	1·6	1·4	5·0	0·2	0	2·4	4·8	100·0

		Starting and finishing.	Loading and raising.	Depositing and steaming.	Standing by.	Locking.	Traffic.	Weather.	Coaling.	Watering.	Moorings.	Repairs.	Miscellaneous.	Total.
Ten steam hoppers, working in docks and channels.	Hours.	35·0	531·0	662·8	208·4	68·0	48·9	38·4	9·1	4·9	121·0	8·7	18·7	1754·9
	Per cent.	2·0	30·2	37·7	11·9	3·9	2·8	2·2	0·5	0·3	6·9	0·5	1·1	100·0

Discussion.

The Author. The AUTHOR, in introducing his Paper, exhibited a number of lantern-slides of dredging plant.

The President. The PRESIDENT, in moving the vote of thanks to the Author, remarked that he had not only dealt with a most interesting subject but had presented a Paper that would be extremely useful both for discussion and for reference. He himself was particularly interested in it because he knew all the ports and harbours that the Author had mentioned. He had worked in the construction of some of them, and was therefore glad to hear of their subsequent history. The engineer and the contractor often left some troubles to be dealt with by the maintenance staff!

Mr. Wentworth-Sheilds. Mr. F. E. WENTWORTH-SHEILDS remarked that in *Fig. 16* (p. 445) the Author showed that the costs of the dredging for which he was responsible had been reduced since 1929 from 4·7*d.* per cubic yard to 2·5*d.* per cubic yard. Such a reduction indicated remarkable improvements in the organization.

On p. 441 the Author stated the total output in tons; that revived a question which had often been discussed, namely, was it possible to adopt a standard unit for dredging? Dredged material could be measured in cubic yards in situ or in barge, and also in tons, and it would be most convenient if one method could be standardized. He had to admit, however, that on the Southern Railway it had not been found possible to adopt one method only. At a port such as Folkestone, where there was sand, it was very difficult to take soundings and to measure in situ, because the site was exposed and because the changes in the level were very rapid. It was, however, comparatively easy to measure sand in barges, because the sand settled down quickly, the measurement obtained corresponding fairly well with the original measurement in the ground. At a port such as Southampton, on the other hand, where a very soft mud had to be dealt with, measurement in barge was most unsatisfactory. Such mud was colloidal, weighed only about 1·0 ton per cubic yard, and in its original state contained about three-quarters of its bulk of water. When that mud was dredged and filled into a barge it did not displace the water which was already in the barge but mixed with it; the resulting mixture, although perhaps only about 5 to 10 per cent. lighter than the original mud, was very much more bulky. Moreover, the increase in bulk varied considerably. Generally speaking, it was found that 1 cubic yard

of mud in situ became about 1·6 cubic yard in the barge. Hence, if the cost of the dredging were found to be 4*d*. per cubic yard in barge, the real cost per cubic yard in situ would be about 7*d*. On the other hand, it was comparatively easy to measure soft mud in situ; soundings could be taken both before and after dredging and the results computed with fair accuracy, provided that the work were carefully done by an experienced surveyor. For the reasons which he had stated, the Southern Railway Company adopted the cubic yard in situ for measuring materials such as soft mud, and the cubic yard in barge for hard materials such as sand. They had abolished measurement by tonnage, as it seemed to have very little value for ordinary purposes. Mr. Wentworth-Sheilds.

The Author had stated that the Great Western Railway employed bucket-ladder dredgers for the bulk of its work; the Southern Railway employed similar machines for most of its work, and he thought that on the whole they were the most useful type, as they were most adaptable and could give very large outputs. An interesting attempt had, however, been made 2 years ago to try to cheapen the dredging at Southampton, where it was very expensive because the barges had to be taken over 25 miles before they could dump their load. He had thought that the mud might possibly be removed more cheaply by stirring it up and allowing it to be washed away, but investigation had proved him to have been wrong. That investigation had, however, given very interesting information. The velocities of the currents, the salinity of the water, the rate of silting per annum, the silt-content of the water and the nature of the silt had all been studied. The latter investigation had been especially interesting, and had been carried out by Dr. Bernard Dyer, F.R.S., an agricultural chemist. He had found that the silt was very fine, 70 per cent. of the particles being less than 0·01 millimetre in size. Although it was sticky, being apparently a kind of very soft clay, it was not really a clay, as 70 per cent. of it was silica and only 5 per cent. was alumina. It also contained 10 per cent. of chalk, presumably because the rivers which flowed into Southampton Water passed largely through chalk country. The most interesting discovery had been that, though when shaken up in distilled water that fine mud settled very slowly (so slowly that after standing for 3 weeks in a glass jar there was still a very large proportion of the mud left in suspension), when it had been shaken up in salt water and allowed to stand the whole of the mud had settled within ½ hour. That showed clearly that it was useless to stir up the mud in a dock where the currents were small, as it would settle again in a very short time. His Company had therefore abandoned the idea of removing mud by that means.

Mr. Wilson. Mr. M. F.-G. Wilson observed that the Author had referred at some length to the accumulation of material at an obstacle projecting from the foreshore, and had mentioned Madras and other places where there had been a good deal of deposit. He had not, however, referred to the erosion on the lee side of the obstruction, which was often much more harmful than the deposit on the weather side. At Madras there had been a great deal of trouble on the lee side of the harbour, where the beach had all travelled to the north, the material from the south, which should have replenished it, having been cut off by the projection of the breakwaters forming the harbour. Much damage had been done, and it had cost a great deal of money to build up the foreshore. Similarly, at Dover the Admiralty Pier had trapped all the material coming from the westward, and the preservation of the beach facing the town, by means of groynes and other works, had been very expensive. At Brighton the beach in front of the town was retained by groynes, but to the eastward it was disappearing, and costly works were now being built in order to protect the cliffs. It would be seen, therefore, that accumulation caused by a projection was not the only resulting problem to be dealt with.

He agreed that it was very difficult for an engineer to foresee the exact effects of interference with tidal currents. The Author had referred to the Mersey bar, and implied that it had been formed by the material brought down by the river. The river Irwell, however, which fed the upper estuary, was very small, and it did not bring down very much silt. The bar consisted of material brought up from the south by the littoral drift, with the result that in 1890, when the bar had first been cut through, it filled up again as fast as it had been dredged, owing to the northward travel of the beach, which accounted for the very large amount of dredging that had been required, as referred to in the Paper. It appeared, however, to be diminishing, and there was now less dredging at the bar than there used to be.

With regard to the Bristol Channel ports, the Author referred to the silt which was brought down by the rivers. He did not think, however, that the silting in the ports was chiefly due to silt being, at the present time, brought down by them. On the Bristol Channel foreshore there were many miles of mud flats, being accumulations of mud which had been brought down during thousands of years; the first wash of the flood tide stirred up that mud, which was then carried by the tidal currents into the neighbourhood of the various docks, where it was deposited at slack water, causing the trouble and inconvenience referred to in the Paper.

Mr. N. G. Gedye desired to call attention to two matters upon which there might legitimately be a difference of opinion. On p. 424, referring to the beach accumulation at Dungeness, the Author mentioned that pebbles from Budleigh Salterton, near Exmouth, as well as Cornish pebbles, had been picked up there, and on p. 425 he suggested that the Dungeness deposit consisted of "sea-borne material from the more westerly littorals of Devonshire and Cornwall." Under existing geological conditions, however, the principal source of supply of the Dungeness shingle was flint derived from the cliffs between Dungeness and Beachy Head. Relatively little came from further west than Beachy Head, and he doubted whether any was derived from west of the Isle of Wight. The late Mr. W. H. Wheeler had been of opinion that the pebbles of remote origin, for instance, those from Cornwall and Devon, found at Dungeness, might have been derived from ballast contained in vessels formerly wrecked in the locality.[1] A list of the "foreign" pebbles found at Dungeness was given in a report of 1895.[2] Mr. Gedye.

On p. 432 the Author, in enumerating the available methods of maintaining waterways, included the "physical removal by dredging or sluicing" of accumulations of sand and shingle, and again at the end of p. 433 he stated that under favourable conditions channels could, at least in part, be cleared by sluicing with impounded water released at or about the time of low tides. He would like to ask the Author whether he could name any harbour where sluicing was now employed with success on any important scale, excepting purely local sluicing to remove accumulations at dock-entrances and in the neighbourhood of dock-gates. The only instance mentioned by the Author where sluicing was employed, except in that very limited manner, was at Port Talbot, which seemed to be a very special case with limited effect, and the Author himself pointed out that the lowering of the dock water-level due to the discharge of sluicing water had to be balanced by pumping in channel water; under such circumstances the expedient could hardly be regarded as economical.

In former times many of the ports on the coasts of Northern France and Belgium, including Calais, Ostend, Boulogne and Dunkirk, as well as similar ports elsewhere, had been provided with large sluicing-basins, but all of them had now disappeared or were disused.

---

1 W. H. Wheeler, "The Sea Coast," p. 200. London, 1902.

2 "The Rate of Erosion of the Sea Coasts of England and Wales, and the Influence of the Artificial Abstraction of Shingle or other material in that Action." Fourth Report of a Committee of the British Association, p. 352. London, 1895.

Mr. Gedye. The tide had been allowed to flow into those basins in order to augment the natural tidal scour. The water had been shut in at high tide by gates or sluices, and had been released at low water, producing a strong current through the channel. The sluicing current, however, had rapidly lost its velocity in passing down the channel, and had only been effective near the basin-outlet and down to a moderate depth below low water. The sand and silt removed by scour had been re-deposited to form a bar or shallow in some other part of the entrance or approach-channel. The President would no doubt recall that at Ostend a few months after the Armistice they had experimented with using the old sluicing-basin there, but had found that material which they removed from the channel had been immediately deposited in front of the one usable quay which had been available for shipping. The introduction of powerful mechanical dredging appliances had led to the abandonment of sluicing-basins, and it would, he suggested, be unfortunate if the Author's references to sluicing were to be read by others, only partly informed of the facts, as implying more than the Author intended.

The figures given on p. 437 relating to the Mersey Docks and Harbour Board's sand-pump dredgers seemed to him to contain a numerical error. The Author stated that one of the sand-pump dredgers " is capable of dealing with some 120,000 cubic yards a day of 10 hours, working in a depth of 70 feet." The largest suction-dredger now employed by the Board in dredging sand from the bar and channels was the *Leviathan*, built in 1909; that vessel had a hopper-capacity of 10,000 tons, and, he believed, filled her load in about 1 hour, but the effective dredging-capacity of the vessel per day of 10 hours could not possibly be as much as 120,000 cubic yards, or even 120,000 tons, for the time occupied in steaming to and from the place of deposit would be more than the actual dredging-time. The four sand-pump dredgers of most recent construction employed by the Mersey Docks and Harbour Board all had a hopper-capacity of 3,500 tons, and were wholly employed in removing sand from the bar and the sea and river channels. The latest of them, the *Hoyle*, built in 1935, lifted 3,500 tons of sand in 50 minutes and had a maximum dredging depth of 70 feet. It should be noted that the average distance travelled by the Mersey suction-dredgers between dredging-positions and the place of deposit at sea was about 7 nautical miles. He quoted these figures from authoritative data supplied to him last year by the Board's Engineer. In connexion with the Author's statement on p. 428 that 9 million tons had been dredged in 1934 from the Mersey bar and channels, he would remark

that that figure had been in former years largely exceeded, the quantity of material removed having sometimes amounted to about 20 million tons in a year. As a result of the training works begun in 1908 and of intensive dredging, the depths in the Mersey approach channels had been increased over considerable areas, although the shallowest or governing depth on the site of the old bar, which in 1907 had been 26 feet at lowest low water, had been still only 27 feet in 1935. Mr. Gedye

It would add to the value of the cost figures given by the Author on pp. 441 and 445 if he would state the amounts which should be added to the total and unit costs in respect of interest on capital, depreciation, renewals, and repairs not already included.

Mr. G. J. Griffiths, referring to sluicing as a means of removing silt, remarked that from his experience he would advise that, unless it was unavoidable, it should not even be contemplated; the remedy was often worse than the disease. The Author justified his own use of sluicing by saying that he only wanted to put the silt somewhere where he could get at it; but how much scour would be brought about? The scour was often more troublesome than the silting that it was intended to cure. Mr Griffiths.

The question of littoral drift was so important that the experience of harbour engineers as to the effects of their breakwaters was always of interest, and he hoped that such engineers would give further information regarding the success or otherwise of the breakwaters erected to safeguard the entrances to various harbours. The quality of the silt was also a very important factor in each case. Did the flood-tides or the ebb-tides carry the most matter in suspension, and how did those quantities compare with that brought down by a river in flood?

An interesting instance of littoral drift was the shingle bank opposite Totland bay. For years it had existed just east of the Needles, extending to Totland bay pier, but last winter it had disappeared; he believed that that had been caused by the gales of last winter, but further information about it would be of value. Referring to Dungeness, some protective works had recently been carried out at Rye, where groynes had been put up to check the littoral drift so as to maintain the harbour entrance; protective works had also been carried out to stop the tide getting through into what was becoming a bungalow town. It would be interesting to know whether the designs of the groynes and other works had proved effective or not.

With regard to the rivers Severn and Wye, during the War he had been in command of Nos. 1 and 2 shipyards at Chepstow, and

Mr. Griffiths. the suggestion had been put forward at one time that there should be a berthing wharf on the left bank of the Wye just at its junction with the Severn. However, it was hopeless to think of berthing ships there; so much matter was brought down by the Wye that a dredger would have to be kept there all the time. He believed the wharf had been constructed, but he doubted whether it had ever been used. The Mersey bar had been mentioned by the Author and by speakers. There was a similar bar on a much smaller scale across the entrance to the Manchester Ship Canal channel at Eastham locks; it was yet another instance of trouble arising at the junction of two streams, and it had not been found possible to prevent it. The experiment had been tried of sluicing the silt down from immediately below the entrance to the locks, but it had not proved effective and had had to be given up because the resulting scour had caused more trouble than the silt.

Mr. Buckton. Mr. E. J. Buckton observed that variations in waterways were infinite, and no two were alike, but they had certain similarities which could be traced and used to practical advantage; the Author gave eight types of such similarities on p. 426. *Fig. 7* showed an unusual case, but there had been a good example of it in Canada. In the estuary of the Nelson river in Hudson bay a solid jetty had been built running out from the shore to the deep channel. However, as the jetty had been built out so the ground had accumulated on each side of it, and very deep scour had taken place in front of it; it had finally had to be abandoned. After that failure an open pier structure had been built, ending in an island of cribwork with breakwaters to form a harbour in the estuary. The resulting conditions were as shown in *Fig. 8* of the Paper, where there was an open arm with a protective structure at the end of it. As a structure, it had been a success, but the scheme had been abandoned because it was uneconomic.

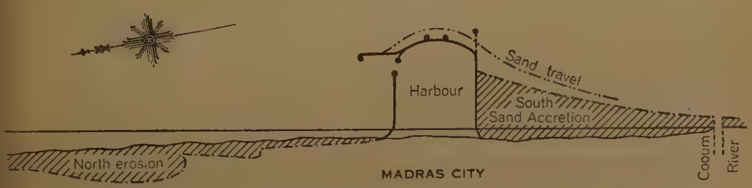
On p. 427, referring to Madras harbour, the Author stated that "siltation commenced even during the construction of the work, and has caused considerable trouble and expense ever since. Unlike Ceará harbour, that at Madras has been maintained by dredging." He did not think that was correct; very little dredging was done at Madras, and the total silting there, if no dredging were done, would be 1 inch per annum. Madras harbour had been built on a shore running north and south, with an entrance facing east. It had no sooner been built than littoral drift began to accumulate against the southern breakwater, and worked round towards the entrance. The coast a few miles to the northward had been eroded very badly, owing to the obstruction of the drift that would otherwise

Mr. Buckton.

have replenished it. At first the harbour had not been a success, because the entrance had been entirely exposed to the waves, which always approached from directions between north-east and south-east. A new entrance had therefore been made on the northern side, the old entrance closed, and a protective arm added as shown in *Fig. 17.* Since then sand had begun to drift along the eastern side; to prevent that action a new arm had been added a few years ago, as shown dotted in *Fig. 17.* For the time being that had lessened the accumulation along the east side, but a stage had now been reached when the next step would have to be seriously considered. At present, therefore, Madras harbour was really kept open by the protective action of the breakwaters rather than by dredging.

The secret of successful marine works was that the engineer ought to work with nature as far as possible. A recent case of the interception of littoral drift had occurred at the new harbour of Vizagapatam,

Fig. 17.



in India, and had been fully described.[1] Vizagapatam was on the same coast as Madras, but the problem there was being dealt with in a different manner. The harbour was inland; the approach-channel had been dredged, and it was partly maintained by the river-action and partly by dredging. There was a heavy northward littoral drift, of the order of 2,000,000 cubic yards per annum. That had been dealt with by sinking two ships and piling stones around them, thus forming an island breakwater under whose lee a dredger could work in a sand-trap. The problem was, what was to be done with the dredged material? It would be cheaper to send it out to sea and dump it, but actually the town of Vizagapatam was north of the channel, and would be endangered by erosion. It had therefore been decided that, to avoid incurring very heavy protective works outside the town, the proper method was to provide a sand-pump dredger and to pump across the channel to the northern shore,

[1] W. C. Ash and O. B. Rattenbury, "Vizagapatam Harbour." Journal Inst. C.E., vol. 1 (1935-36), p. 235. (December, 1935.)

Mr. Buckton.

working with the natural process. Another interesting example of trying to work with nature was the maintenance of the port of Chittagong. The river Karnafuli had formed a sharp bend about 4 miles from its mouth, and in an ordinary river, such a bend would be expected to go on increasing in severity until a cut-off occurred. Actually, however, there was a very strong tidal action, and the ebb-tide tended to draw the current of the river by the shortest path—that was, by the convex side instead of the concave side. A training work had therefore been required on the convex side to stabilize the channel. The action of natural forces on waterways differed in every case, and every case had to be studied individually. The Author dealt with the Great Western Railway's ports in South Wales; they were all on the same tidal river and yet they differed greatly. Whilst some of the harbours and docks of the world were on tidal rivers, others were on oceans, inland seas, great lakes and non-tidal rivers, and the briefest summary of their conditions could fill a Paper in itself.

Mr. Du-Plat-Taylor.

Mr. F. M. G. Du-Plat-Taylor referred to the maintenance of the approach-channels to docks by the use of sea-going sand-pump dredgers. The outstanding instances of the use of those vessels had been at the ports of Liverpool and New York, and more recently at Bordeaux. All such vessels were self-propelling and self-loading, and were not adapted for loading independent hopper-barges. Almost the only loss of working time was that due to weather, so that the vessels had to be large and seaworthy, and the suction-pipes and connexions had to be designed for working in a considerable seaway.

The original dredgers on the Mersey bar, as well as the New York dredgers, had embodied a number of patents of the late Mr. A. G. Lyster, Past-President Inst. C.E., at that time Engineer-in-Chief of the Mersey Docks and Harbour Board. One of the first of those vessels, the *G. B. Crow*, had a hopper-capacity of 3,000 tons of sand and could load itself in 40 minutes. The suction-pipe was 45 inches in diameter; it was housed in a well amidships, and could work to a dredging-depth of 47 feet. The pipe sloped aft in the dredging position, but that had proved to be a mistake, as, in the event of the anchors dragging, the pipe-nozzle was forced into the ground and the pipe sometimes buckled or the joint at the top fractured. In later dredgers the pipe or pipes were fixed outside the hull and sloped forward. The hoppers were provided with hydraulically-operated cylindrical valves instead of hopper-doors. The load was often so solid that a man could walk about on it, and water had to be forced into the hoppers through special pipes to assist

discharge. These dredgers could work in a beam sea, with waves of a height of about 5 feet from crest to trough, the pipe-joint allowing sufficient lateral movement, but in a head sea there was risk of buckling the pipe. The largest dredger in the Mersey, the *Leviathan*, had a capacity of 10,000 tons raised in 50 minutes, and was 465 feet 9 inches long between perpendiculars, 69 feet in beam, and 30 feet 7 inches deep. It had two suction-pipes fitted outside the hull and sloping forward. Those vessels maintained a channel 1,000 feet wide, originally to a low-water depth of 29 feet, but now to 30 feet. Before dredging had been begun the low-water depth over the bar was only 11 feet. In addition to the dredging, a submerged training-wall or revetment had been constructed to direct and confine the currents in the low-water channel. Mr. Du-Plat-Taylor.

At the entrance to the Port of Bordeaux a deep-water channel with a low-water depth of 36 feet and a width of 3,280 feet was being dredged through the bar of the Gironde by sand-pump dredgers. The weather conditions on that bar were very severe, as it was exposed to westerly and south-westerly gales from the Atlantic, and it was necessary to employ vessels which could work in heavy seas. With that object, flexible suction-pipes had been adopted, with ball-joints at the top and with three intermediate flexible joints of the Guilloux or of other types. The pipes were fitted outside the hull, the suction-nozzle being dragged astern while the vessel moved slowly ahead. The largest dredger, the *Pierre Lefort*, had a daily output of 30,000 cubic metres and a hopper-capacity of 3,132 tons. The vessel was 337·9 feet long, 54 feet in beam, and 19 feet in loaded draught, and dredged to a depth of 65 feet, being able to work in seas with waves of 11 feet 6 inches from trough to crest. It seemed probable that future deep-sea sand-pump dredgers would develop along those lines.

Mr. Robert Chalmers hoped to elicit from the Author some further information as to the reasons for his statement, on p. 438, that trials of sand-pump dredgers "including those with the more efficient drag-suction dredgers, were not carried out, as it was not anticipated that they would result in any improvement in economic working." He himself had had some experience, under the President, of dealing with the material at Newport, Mon., in connexion with the later stages of the construction of the new lock-entrance. The local material had been dredged with bucket-ladder dredgers, and, at one stage, the contents of the barges had been pumped ashore with the idea of raising the level of certain low-lying areas around the docks. Those low-lying areas had formed a settling-pond; the surplus water, after leaving it, had passed through another Mr. Chalmers.

Mr. Chalmers. very large low-lying area, which had constituted a further settling-pond, and had then been discharged into the timber-float which was shown in *Fig. 10.* Those operations had had to be stopped because, in spite of the settling-ponds, the timber-float had silted up and had had to be dammed off, pumped out, and the silt excavated. Not only that, but the timber-float was, or had been, connected with the North Alexandra dock by a small canal in which was the intake of the power-station that supplied water under pressure to the hydraulic cranes around the quays; the dock management had complained that the silt in that water was spoiling the valves of the hydraulic cranes. He and his colleagues representing the contractor had been very sceptical, but the case had been so well established that they had had to pay for the damage caused. That was an illustration of the character of the material at Newport, Mon., and also, he submitted, of the waste and inefficiency involved in dealing with it by any method which would involve adding any more water to it than could possibly be helped. In 1922 he had been sent to dredge mud out of Dover harbour in the area between the Prince of Wales's pier and the Marine station, and had employed a bucket-ladder dredger, and hopper-barges of a capacity of 600 cubic yards each. He had started by loading those barges with 566 cubic yards each, barge measurement, which, as the bulking factor was $18\frac{1}{2}$ per cent., was equivalent to 476 cubic yards in situ. When he had sent about 100,000 cubic yards to sea his soundings had told him that about 12,000 cubic yards had been redeposited on the area already dredged, and he had had to cut down the loads in the hoppers to 470 cubic yards, barge measurement, equivalent to 396 cubic yards in situ. That again showed that there was waste and inefficiency in processes which involved adding water to the material, and it was impossible to avoid doing that if a bucket-ladder dredger were employed. In contrast, he had seen a modern drag-suction hopper dredger fill its 1,200-cubic-yard hopper in less than 10 minutes with mud so thick that a spade would stand upright in it, and containing in fact only 15 per cent. added water as compared with its original condition on the bottom. The dredger had had to travel $1\frac{3}{4}$ mile to the dumping-ground and to return to start dredging again, but it had completed the whole cycle in 50 minutes, whereas his own hopper-barges at Dover had taken over an hour each to fill alone. He had estimated that the cost of the work he had seen done with the drag-suction hopper dredger was probably less than half the cost of the same work if done with a bucket-ladder dredger and hopper-barges.

*** Dr. BRYSSON CUNNINGHAM observed that the Author's remarks on the effects of the intrusion of structures into the sea from the shore-line deserved serious consideration by all harbour-engineers. Any disturbance of the normal regimen of a coast was bound to be attended by important consequences, in particular, the accumulation of material in positions where it might be detrimental to the maintenance of harbour-approaches and river-channels. There was another aspect of the matter, moreover, which should not be overlooked in that connexion, although in the Paper it was somewhat obscured by the main consideration. In describing the accretion which followed the construction of solid structures projecting from the shore in sandy localities, the Author did not specifically direct attention to the fact that accretion was generally much more pronounced on one side of the obstruction than the other, nor was mention made of the tendency of the littoral current to produce, on the leeward side of the obstruction, erosion which might have serious results. Without definite explanation, *Fig. 7* did not bring out that important fact; no doubt, the Author was considering his subject solely from the point of view of the removal of unwelcome accumulations. At the same time, the eddying effect of the littoral current could not be disregarded, and it was to be noted that it gained in intensity if the pier, or groyne, had a returned end or curved extremity.[1] Confirmatory evidence was forthcoming from experience in various places, notably from Madras harbour, which was mentioned in the Paper. Following the construction of the breakwaters, erosion had extended northwards for a length of several miles along the coast, and had had to be checked by means of stone revetments.[2] As another instance, he had been consulted about 2 years ago regarding the serious loss of sand from Montecito beach in Santa Barbara county, California, U.S.A., where by the construction at an outstanding point of a pair of groynes, 200 feet and 250 feet long respectively, the leeward coastal frontage, formerly a fine sandy beach eminently suitable for bathing, had been denuded of sand for a distance of about 500 yards from the groynes, the level of the beach being lowered several feet until checked by a harder stratum consisting of boulders and shingle. To the windward of the

Dr. Cunningham.

*** This and the succeeding communications were submitted in writing.—SEC. INST. C.E.

[1] Brysson Cunningham, "Harbour Engineering" (3rd edition), pp. 33 and 34. London, 1928.

[2] Sir Francis Spring, "Coastal Sand-Travel near Madras Harbour." Minutes of Proceedings Inst. C.E., vol. cxciv (1912–13, Part IV), p. 153.

Dr. Cunningham.

groynes, a beach of sand had been built up. Accretion and erosion, in fact, were complementary phenomena, and both could be very troublesome to the harbour engineer. In the case of certain waterways, measures to counteract erosion were as necessary for their maintenance as measures in other places for the removal of excessive accretion.

At the beginning of his Paper, the Author seemed to stress the application of his remarks to tidal waters, as if tidal conditions were essentially associated with, or mainly responsible for, the processes he described. That, if a correct inference, was hardly the case. There was marked littoral drift and change of shore-contours along the coasts of practically tideless seas, for example in the Black Sea and the Gulf of Mexico, whilst flood-waters from the mountains of Central Europe encumbered with silt and detritus the mouths of Mediterranean rivers (where tides were also negligible), as illustrated by the deltas of the Po and the Rhône. The term "silting" appeared to be used in the Paper to cover both the deposit of silt, properly so called, and the transport of coastal material. The distinction was important, since silt, or mud, and fine sand were capable of being carried in suspension by water, whereas the heavier material, such as pebbles, shingle and gravel, did not float but was moved along the shore, being largely set in motion by wave-action. The accretion at Dungeness, for instance, was almost entirely of shingle, and could not be classified as silt, nor its deposition as "silting." Dr. Cunningham was hardly prepared to agree that bars at river mouths were due to "large quantities of alluvial matter eroded from the land" and brought down by rivers (p. 428). Indeed, the illustration of the Mersey bar in that connexion seemed a little unfortunate, since that bar consisted mainly of sand of varying degrees of coarseness, and not of alluvium or silt.[1] In his opinion, the origin of bars was to be sought chiefly in the travel of littoral drift, which was partially arrested when it came into contact with a transverse current issuing from the mouth of a river.[2]

The latter part of the Paper, dealing with dredging operations at the Great Western ports, contained much interesting and useful information. Several questions suggested themselves, but he would only ask whether any special steps were taken to secure the retention of the fine silt in the hoppers and prevent its wasteful escape overside. Had a trial been made of the adjustable coamings used on the

---

[1] There was some admixture of mud on the outer slopes: the coarsest and cleanest sand was found on the inner slopes (A. G. Lyster, "Dredging Operations on the Mersey Bar," British Association Meeting, 1895).

[2] Brysson Cunningham, "Harbour Engineering," p. 347. London, 1928.

Liverpool hoppers, as devised and described by the late Mr. A. G. Lyster, Past-President Inst. C.E., Engineer-in-Chief to the Mersey Docks and Harbour Board ?[1] It would be interesting to know if the good results obtained in the Mersey had been realized elsewhere. Dr. Cunningham.

Mr. J. N. DAWE observed that the Author referred on p. 433 to the disadvantages, in some cases, of solid training works, and the success of lighter structures. Further examples would have been very useful; in that connexion, the case of the lighter-port of Pamanoeken on the north coast of Java might be of some interest. The river Tjipoenagara on which that old port was situated maintained a good channel as far as the foreshore of the delta, where it had become so silted-up that mud-flats extended about ⅔ kilometre or more from the edge of the maritime jungle, where the navigable channel ended. Beyond that the channels were tortuous and irregular, and the low-water depth at the bar had become reduced to about 25 centimetres, resulting in practical disuse of the port. A scheme had been drawn up for lighter docks to be built and dredged out a few miles distant in the middle of a long beach, as training the estuary of the river had been considered out of the question, owing to the high cost of stone locally. About 10 years ago he had been asked to advise on that scheme, and had said that, in his opinion the proposed lighter dock would need almost constant dredging, owing to littoral drift of sand, but that the estuary could be made navigable for lighters and small craft by works of a temporary or semi-permanent nature and low cost. Such works had been put in hand a year or two later, and followed the principle recommended by Mr. Buckton of assisting nature. Among the cheapest materials available were bamboos. Two lines of fences, composed of canes driven into the mud and sand with clear spaces of 2 to 4 inches between them, were run out about 160 feet apart, towards the sea. What was considered the most suitable direction for a straight channel was chosen irrespective of existing channels or sandbanks. Where channels of any depth were crossed, the fences were strutted by bamboos driven at an inclination and lashed with palm-fibre ropes to the walings, also of bamboo. Where banks protruded between the fences, short groynes of the same materials were used to direct the current and expedite erosion. Where wave-action was considerable, the fences were strengthened by driving king-piles and struts of young teak trees, and additional fences or groynes of shorter length were made at an angle to check the waves and littoral drift caused by the east monsoon, during which most erosion and drift Mr. Dawe.

[1] Annales des Travaux Publics de Belgique, vol. v (October), 1898.

Mr. Dawe. took place. These, being of open construction, caused sand to deposit on their lee sides. The effect was that, during the first wet season, banks of silt were formed close up to the outsides of the two main fences as far out as they were driven, cross channels of depths of 12 feet being filled sometimes by a single flood. Those banks were consolidated by seedling trees of suitable kinds, some planted and some self-sown. In 2 years there was a thick growth of scrub on each side of the channel, and there was no necessity to maintain further the first part of the fences. The river scoured out its own channel to a sufficient depth to allow of the required navigation. The works had been extended each year to a diminishing extent as deeper water was reached, and when experience of the local conditions had shown the best positions of the secondary fences or groynes, a channel navigable at all stages of the tides had been, and still was, maintained throughout the wet and dry seasons, its length being about 2 kilometres from the original starting point. A few other precautions had had to be taken, but the total cost had been extremely low, and no dredging had been necessary.

Mr. Hindmarsh. Mr. R. F. Hindmarsh, referring to the Author's statement that wherever an engineer created a harbour in tidal waters, thereby causing interference with the littoral drift, he would sooner or later have to take steps to preserve the necessary depth of water for safe navigation, stated that one of the main objects in view in the design of the works at the river Tyne entrance had been the removal of the old bar, which, before the construction of such works, existed across the entrance to the river and had over it only about 6 feet of water at L.W.O.S.T. There was now a depth of 30 feet at L.W.O.S.T. and little if any dredging would be required to maintain that depth if it were not necessary to maintain a straight entrance-channel; the main current on both flood and ebb tides, however, followed a curved track to the south, and a shoal formed on the north side of the channel which was removed by dredging for a few weeks every 2 or 3 years. The littoral drift was from north to south, and, although the breakwaters extended on the north side ½ mile and on the south side 1 mile from the original shore-line, accretion had only occurred on the south side of the south breakwater, and no interference with navigation had been caused by that accretion.

Mr. Latham. Mr. Ernest Latham found the Paper of particular interest to him, especially in regard to those portions dealing with silting questions at riverine harbours, and also because the Author referred to the Paper by his late Partner, Mr. A. E. Carey, on the sanding-up of tidal harbours. Since the reading of Mr. Carey's Paper, however, as the result of experience he had ceased to agree with some of the views

expressed therein. For example, he would not agree to *Fig. 7* as being a proper representation of the arrest of littoral drift by a groyne. His own experience was that on the lee side of most groynes there was nearly always definite erosion to a much greater extent than shown in that diagram, except under the temporary reversal of littoral drift due to changes in the direction of the wind. However, he entirely agreed with *Fig. 4*, as it represented in diagrammatic form the cross-channel port of Newhaven, which was notably safe, and where the construction of the outer breakwater seemed to have been an entirely successful means of keeping the entrance clear with a minimum of dredging; the port could generally be used by cross-channel steamers at periods when the larger ports of Dover and Folkestone were temporarily out of commission owing to heavy weather. Mr. Latham.

The Author's statements in regard to the conditions at Dungeness had probably been true 10 years ago, but did not properly describe the position to-day. The littoral drift passing into Rye bay from Hastings round Fairlight cliffs had now almost entirely ceased, and it had been the recent starvation of Rye bay's natural shingle supply that caused the breach at Winchelsea in 1933 by inundation of the sea. The consequent expenditure on sea-defence works carried out by him on that site for the Catchment Board concerned reached nearly £200,000. Further, instead of accreting at the moment, the Dungeness formation was being rapidly eroded on the western side, and the "Ness" was tending to form a hook towards Sandgate. Even Littlestone, on the eastern side, had lost much shingle recently, and the foreshore had been lowered by several feet, necessitating the entire reconstruction of about 1 mile of the existing sea-defences there.

The dredging by the Mersey Docks and Harbour Board in Liverpool bay was, of course, a classic example of heavy costs in maintaining the entrance to a riverine port. In 1927, when advising the Lancashire County Council on the question of coast-erosion in Liverpool bay, he had had occasion to study the matter, and the Author would no doubt be interested to hear that the quantity dredged was as high as 17 million tons in 1909. The figure had subsequently fallen to an average of 4 million tons per year, so that it would appear from the Author's figure of 9 million tons in 1934 that siltation was again on the increase in Liverpool bay.

The Author did not apparently refer specifically to any trouble at the small harbour of Burry Port, now owned by the Great Western Railway Company. Immediately before the War the original owners of the harbour, the Burry Port and Gwendraeth Valley

Mr. Latham. Railway Company, had been very much concerned with the silting up of Burry Port, and from surveys he had made on their behalf the harbour at that time had appeared to be in danger of total eclipse. From such records as were available it appeared that the Burry inlet had three channels passing through vast accumulations of sand. One of them was always the main channel, but from an examination of records a change of channels seemed to occur about three times in a century; at the time of his investigations, the north channel had been closing and the middle channel had been opening, tending to leave Burry Port more or less high and dry on the north side of the inlet. In 1914 the rate of accretion immediately to the south-west of the harbour entrance along a foreshore length of 1,600 feet had been as much as 50,000 cubic yards per year. Dredging against accretion on that scale had seemed rather a hopeless proposition for so small a harbour, and an extension of the western breakwater had been recommended. The Company, however, had come to the conclusion that they could not spend the sum involved, and it would be of interest to know what was the present position at Burry Port, and whether the Great Western Railway had extended the western breakwater in the manner suggested.

With regard to siltation, one point had now become of importance in England; namely, that for land-drainage purposes the new Catchment Boards functioning under the Land Drainage Act of 1930 were doing much up-river dredging nominally for drainage purposes, although actually it frequently resulted in improved navigation to small inland ports. The effect on the ports at or near the entrance of the river was adverse in cases where vessels took the ground at low tide. A present instance of that effect was the difficulty being experienced at Sutton Bridge, near the entrance of the river Nene into the Wash, where the increased flow of ebb and flood tide due to the increased up-river tidal compartment caused by dredging appeared definitely to have disturbed shipping berthages just below the Sutton Bridge swing-bridge. At Sutton Bridge there was an example that should be a warning to all maritime engineers. Towards the end of the last century large sums of money had been spent on excavating a large dock in the Fens, with approach-works into the river Nene. The dock had been opened to shipping for 2 months only, and as the dock company could not cope with the dredging, the site was now completely filled up and formed a golf-course.

Mr. Richards. Mr. B. D. Richards observed that the littoral drift that occurred on the Dead Sea might be of interest. He had recently carried out investigations in connexion with proposed development-works there,

and as the latter included small harbours or jetties at both ends, the question of littoral drift was important. The Dead Sea was 45 miles long, with a maximum width of 10 miles, and lay north and south. It was divided by the El Lisan peninsula into two arms, of which the southern, 15 miles long, was very shallow. The sea received the inflow of the river Jordan and of several large wadis, but there was no outlet, and the inflow was disposed of by evaporation, giving rise to periodic variation of sea-level. The last peak level occurred in 1929, since when there had been a fall of about 10 feet. There was also an average seasonal variation of 2 feet, the maximum occurring during or after the winter rains. Shingle-beds occurred along the western and northern shores; the origin of the shingle was evidently the disintegrated limestone of the western hills and the stones in the marl-beds surrounding the sea, and it was brought down in considerable quantity by heavy floods in the wadis. The prevailing winds were northerly and southerly; coinciding with the lie of the sea, they caused a southerly drift at the south end and a northerly drift at the north end, and the latter drift carried the shingle along the west shore and round the north shore. It was noticeable that the shingle tended to form regular beaches which in some places projected as spurs into the sea. With a falling sea, those beaches showed ridges marking the high-water level of the preceding winter. The littoral drift being due to wind action, it seemed probable that those spurs originated from deflexion of the wind caused by irregularities in the line of the coastal cliffs. Once a spur had commenced to form, its growth was very rapid, as was shown by the fact that a small jetty at the south end, built out from one of those spurs, was, though of open construction, completely silted-up in a short time. At the north end, with longer fetch and greater depth of water, there was greater wave-action than at the south end, and 6-foot waves had been measured on the north shore. That caused a considerable travel of shingle along the north-west and north shores. Various jetties in that district, of open construction, showed no heavy accumulation of shingle against them, but a partially-closed structure had acted as a groyne and the shingle had banked up against it. Owing to the fact that the Dead Sea was tideless and that the level had been steadily falling for several years, it afforded a good opportunity of studying the subject of littoral drift under wind-action.

Mr. Richards.

The AUTHOR, in reply, remarked that Mr. Wentworth-Sheilds had raised the most important question arising from the Paper—that of a standard unit or method of measurement for dredging. The ton unit was that required by the Great Western Railway

The Author.

The Author. Company's accountants—possibly because of its extremely remote relationship with that familiar railway cost unit, the "ton-mile." The reduction in working-costs, achieved mainly by centralization, was indicated in the Paper. Dredging, carried out departmentally, had as its main object the conservation as economically as possible of navigable depths, the actual method of measurement being immaterial. Should such work be undertaken by contract, however, the circumstances were entirely different and the method of measurement became of supreme importance. In the Author's opinion, whilst no hard-and-fast rule could be laid down, the subject was of sufficient importance to justify some investigation by The Institution as to the possibilities of standardization of measurement of material to be dredged, whether "in situ" or "in barge." The Great Western Railway Company was at present carrying out investigations with the object of establishing more closely than at present the relationship between accurate measurements in barge and in situ.

In regard to the points raised by Mr. Wilson, he had indicated in the Paper the effects of disturbance of the natural balance of quantities occasioned by the construction of harbour works, involving accretion of material in excess of requirements at some places, and reduction in the supply of such material at other places where it was necessary to maintain the stability of harbours and their entrances. Although the Paper was mainly concerned with the maintenance of navigable depths at the ports discussed, the leeward side erosion, although not actually affecting those ports to any material extent, was a question of extreme importance. As an example, the construction of the South Devon Railway at the base of the cliffs between Dawlish Warren and Teignmouth had undoubtedly checked erosion of the foreshore, and had adversely affected the foreshore of the estuary of the river Exe from Dawlish Warren to Exmouth, where erosion was in excess of accretion and was at the present time accumulative. Harbour engineers fully realized that where accreted material was artificially collected (for example, by groynes or breakwaters) some reduction in the supply of material would occur elsewhere. It was quite impossible to deal adequately with that subject in such a Paper as the present.

It was evident from Mr. Gedye's remarks that there were differences of opinion as to the source of accretion at Dungeness. The source of the accretion in a particular case was not, however, of such importance as the principle that any projection from the foreshore would intercept littoral drift.

In general, sluicing could only result in local movement of silted material to sites from which it could be more economically removed.

The Author.

The only G.W.R. port where really useful results were obtained by sluicing is Port Talbot. At Burry Port, however, where the shipments were not comparable with those at Port Talbot, useful sluicing work was done, although the old sluicing pond had for some years been out of action.

He agreed with Mr. Gedye that the effective dredging-capacity of the *Leviathan* was less than that given on p. 437.

He had purposely refrained from giving particulars of other than working costs, including repairs under £100, but had indicated the relationship, so far as his Company was concerned, between working repairs and heavy repairs and overhauls. All-in costs dependent on depreciation, interest on capital, and other factors could not usefully be standardized.

He agreed with Mr. Buckton that the alteration in the position of the entrance to Madras harbour and the construction of the sheltering arm on the east side of it had removed for the present, and probably for a very considerable time,[1] anxiety in respect of siltation. Prior to that alteration, dredging had been essential, and without it, to quote Sir Francis Spring, "it might conceivably have had to be closed in heavy weather to vessels of deep draught, before the new entrance could be opened."[2] Mr. Buckton had emphasized what he had said about the inevitable effect of constructing artificial harbour-works. His statement that "the engineer ought to work with nature as far as possible" constituted the fundamental principle of harbour work, and should be in the mind of every harbour engineer.

Whilst agreeing with Mr. Chalmers that sand-pump and drag-suction dredgers would no doubt deal economically with the silt at Newport and probably also at Cardiff, he was doubtful whether corresponding economy would result from the use of that type of dredger at Port Talbot and Swansea. A main factor in the reduction of dredging-costs in the South Wales ports had been interchangeability of the craft employed, and, although reduction in costs at one port might be made by the utilization of a particular type of plant, the present reductions of cost consequent upon interchangeability would probably out-weigh local economies.

In reply to Mr. Latham, the Author had already referred to sluicing at Burry Port. His Company had not carried out any constructional works there, but the amount of siltation had been insufficient to

---

[1] See also Sir F. J. E. Spring, "Coastal Sand-Travel near Madras Harbour." Minutes of Proceedings Inst. C.E., vol. cxciv (1912–13, Part IV), p. 163.

[2] "The Remodelling and Equipment of Madras Harbour." *Ibid.*, vol. cxc (1911–12, Part IV), p. 96.

The Author. affect the handling of traffic to any considerable extent. Conditions in the estuary were continually changing owing to shifting of sand-banks, and during the last 10 years special steps had been taken to protect the railway between Llanelly and Ferryside. Those changes, and the protective works necessitated by them, were in themselves of sufficient importance to justify the preparation of a special Paper dealing with them.

Paper No. 5059.

"The Diversion of the Main Line between Mottram and Dinting, London and North Eastern Railway."

By Felix William Slade, M.A., Assoc. M. Inst. C.E.

(Ordered by the Council to be published with written discussion.)[1]

TABLE OF CONTENTS.

Introduction.

In order that goods and mineral traffic running daily over the down main line of the Great Central section between Penistone and Manchester, which is the sole means of access for the London and North Eastern Railway to the important manufacturing towns in Lancashire and Cheshire, could be dealt with expeditiously and economically, a new marshalling yard had to be built between Dinting and Mottram. The layout of this yard involved the diversion of the old main up and down lines for 70 chains (the maximum diversion being 200 feet) so as to enable the new yard to be placed on the down or south side of the main line.

The Derbyshire County Council decided on the widening of the existing Glossop Road viaduct, which was also lengthened under the new scheme, whilst the old bridge over Botany Lane was widened under the sidings and a new bridge built alongside to carry the diverted main lines. The scheme also called for fencing, gates, culverts, drains, new footpaths, and a timber wharf-wall for a cattle dock.

The L.N.E. Railway Company's Act of 1930 authorized the work to be carried out, and a contract tender was accepted on the 3rd May of the same year. The work was to be completed in 22 months, but this time was exceeded by nearly 3 years owing to the difficulties encountered in tipping the main-line diversion embankment. This involved the reconstruction of an existing culvert and building two retaining walls at the foot of the embankment. Another existing

[1] Correspondence on this Paper can be accepted until the 15th May, 1937, and will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

culvert near Old Dinting goods-yard was also rebuilt. These works were not allowed for in the original estimate, and they considerably altered and delayed the progress of the work.

Earthwork and Culverts.

The scheme provided for the diversion and shortening of the main line between 10 miles 16 chains and 10 miles 70 chains, the distance being reckoned from Manchester towards Sheffield. The maximum diversion was approximately 200 feet (Fig. 1, Plate 1). The scheme included an embankment from 10 miles 16 chains to 10 miles 50 chains and a cutting from there to 10 miles 70 chains at the junction with the old line. At 10 miles 28 chains the line crossed over an occupation road called Botany Lane, over which a new bridge was built, whilst the marshalling-yard scheme at the side of the main lines required the old bridge over the road to be widened.

The excavation was in boulder clay with irregular beds and pockets of sand and gravel. The embankment from 10 miles 16 chains is 15 feet high and presented no special difficulties in tipping, but from Botany Lane to 10 miles 46 chains the tipping to form the embankment is 90 feet from top to toe at 10 miles 33 chains, although the maximum tip was 60 feet on the natural surface, which sloped downhill towards the north. This slope was very sodden and rubble drains with some pipes were laid to intersect the surface. Tipping with excavation from the up-side cutting was started on the 9th May, 1931, at the toe of the embankment at 10 miles 33 chains, where the original embankment resembles a half-basin with its base in the stream which is taken in a culvert under the embankment. Owing to this weak place at the toe of the embankment, it was decided to build a retaining wall which consisted of mass-concrete ($6\frac{3}{4}$ to $2\frac{1}{4}$ to 1) reinforced with old rails and with rubble pitching behind the wall. The wall was built between October and November, 1930, and the tipping to the embankment with Manchester Ship Canal wagons was proceeding slowly but steadily at that period. The original slope was approximately 5 to 1 owing to old slips having occurred, and the proposed slopes of the new embankment were to be 1 in $2\frac{1}{2}$ for the top 25 feet of depth, then 1 in 3 for the next 30 feet of depth and finishing with 1 in $3\frac{1}{2}$ at the base. A side ditch was constructed on the north boundary from 10 miles 33 chains to 10 miles 46 chains to take the drainage water from the new cuttings. The section of this ditch was open, with a flat invert 2 feet 6 inches wide and with sides battered $\frac{3}{4}$ to 1, approximately 2 feet deep. The invert and walls were cement-joined rubble.

On the 24th August, 1931, the toe wall showed shear cracks extending across the breadth of the wall (Fig. 2, Plate 1), whilst the

culvert arch and side walls, which were incorporated in the wall and formed an outlet for the culvert, were also seen to be fractured. On the 2nd September, the immediate toe of the old main-line embankment was noticed to have settlement cracks between 10 miles 34 chains and 10 miles 36 chains, and on the 15th October the near-side rubble wall and invert of the side ditch were tilted upwards and outwards. On the 8th September two cracks appeared in the culvert and these extended across the breadth; further cracks appeared on the 10th September and on the 10th to 12th November, and the movements increased rapidly, resulting in the ultimate failure of the wall. It was not considered safe to continue tipping and on the 13th February, 1932, the work was stopped and borings were taken at the toe of the embankment at the lowest point, namely, between 10 miles 32 chains and 33 chains, to ascertain the underlying strata; five boreholes revealed intermediate beds of clay and shingle with millstone grit, at about 40 feet below the surface. A section of the strata passed through is shown in Figs. 3, Plate 1.

It was decided to drive interlocked steel sheet-piling about 12 feet in front of the old toe wall and to fill in between the old toe wall and the sheet-piling with mass-concrete ($6\frac{3}{4}$ to $2\frac{1}{4}$ to 1). The piling was to be 15-inch by 5-inch by 39·51 lb. rolled-steel joists, with interlocking pieces, in lengths of 40 feet and 35 feet. Driving was started on the 23rd March, and by the 7th May one hundred and forty-five piles were driven to rock, there being one hundred and seventeen in the front row and twenty-eight used as anchorages.

The existing culverts under the old embankment were found to be of millstone grit, quarried locally, rough dressed on the face with the joints set flush. The stones were set in lime mortar. On cleaning out, the sections were found to be alternately large and small. The large sections have arched sides, 5 feet 6 inches deep internally, and 4 feet $6\frac{1}{2}$ inches wide at the base with a dished invert of 7 inches, whilst the small sections are circular in shape with an internal diameter of 2 feet 9 inches. The plan of these sections is so haphazard and the longitudinal section shows such irregular gradients that it indicates that the culverts were repaired in great haste after a slip.

Before the old embankment was raised it was decided to construct a new culvert to replace the uneven section of the old one and to join with the old culvert at the end of the upper large section whilst the lower end would adjoin the by-pass to the reservoir. The excavation for the culvert was in clay in open cut until a depth of 27 feet was reached, when tunnelling commenced. The culvert outside was constructed with a concrete foundation and brickwork invert, vertical side-walls and a semicircular brick arch, with con-

crete backing to the arch. Work was started on the 7th February, 1931, and the invert was finished up to the heading by the 25th March. On the 13th April a start was made to drive the heading. The width of the heading-excavation was 11 feet at the top and 12 feet 6 inches at the base, and 12 feet 3 inches in depth. The culvert dimensions were 4 feet 6 inches wide by 5 feet 6 inches deep. The foundation was concrete ($6\frac{3}{4}$ to $2\frac{1}{4}$ to 1), and the 9-inch deep invert, side walls, arch and backing were constructed in brickwork. The progress of the completed culvert averaged only from 12 to 14 linear feet per week, although double shifts were worked for 7 days a week. The slow progress of the work was due to the unstable nature of the clay, which necessitated heavy timbering and finishing the building directly behind the tunnel excavation. 9-inch by 9-inch scantlings were first used in the timber head-trees, but these were found to fail by bending, and so the size was increased to 12-inches by 12-inches with satisfactory results.

A shaft 12 feet square was started on the 13th May, 1931, 99 feet from the beginning of the heading, and by the 3rd June the heading had been driven as far as the shaft which had been excavated to the depth of the heading, 38 feet below the natural surface of the ground. At the same time as the heading was proceeding, a factory, adjacent to the railway, demanded a relief culvert around their works, as it was maintained that a larger flow of water would be let through the new culvert, whereas previously the flow of water was bottled by the small section of the old culvert, and that during times of floods this might cause a failure in the culvert which ran under the factory. This work was completed on the 29th September.

The tunnel-heading was proceeding steadily and between the 17th and the 28th September junction was made with the old culvert, and by the 11th October the diversion was completed. After this was done the concrete corbel-slab at the shaft was cast on top of the culvert, and a brick-faced manhole shaft of 4 feet 6 inches internal diameter, and 1 foot $1\frac{1}{2}$ inch thick, was built on top and carried up to the old ground-line. Concrete backing filled up the space at the back of the shaft. It was intended to bring the shaft up to the finished top of the new embankment, and so the shaft was raised 36 feet 6 inches in brickwork above the concrete-backed portion. On the 6th February, 1932, however, the top 19-foot 9-inch length of shaft was tilted 1 foot 4 inches out of plumb (the tipped surface being 9 feet 6 inches below the top), and this movement increased as the tipping to the new embankment approached nearer, ultimately resulting in the complete failure of the shaft, which was sheared in three places.

On the 4th March, the base of the shaft was sealed with a concrete

slab, reinforced with old rails, and the upper part was filled with clay. The culvert still showed radial cracks and these were grouted frequently, the ultimate total amount of creep exhibited by fractures being 3 feet 7 inches. The culvert in heading appeared to have sufficient bearing, although the longitudinal strength relied only on brickwork. As the settlement and slip developed, the apparently stable ground became unstable, thus causing the fractures in the culvert due to the lack of longitudinal reinforcement.

The tipping to the embankment proceeded with lifts of not more than 10 feet, and the toe of the embankment continued to push over the rubble pitching at the back of the reinforced toe wall, although the wall held firm, and eventually the clay approached near to the culvert-outlet. On the 21st March, 1933, 18-inch diameter reinforced-concrete lap-jointed pipes were placed in the open by-pass to the reservoir to ensure that the culvert still had an outlet, should clay fall over the culvert-mouth. As the settlement and slip of the embankment still continued, a timber cover was placed over these pipes on the 30th March, and eventually the culvert was extended in concrete between the 3rd May and the 6th July, and again between the 11th October and the 24th November, when a slab was concreted over the sluice-way with the penstock, which was still controlled by the factory. The sluice-way was extended with the 18-inch diameter reinforced pipes which were used temporarily in the by-pass, and connected to a catch-pit which was placed so as to trap the silt brought down by the drains from the embankment.

The tipping to the embankment proceeded slowly but steadily and the settlement and slip continued. All surface cracks were kept punned and the surface drained, and from the 6th April to the 10th August, 1933, a night watch was kept on the embankment. The side ditch at the toe of the embankment failed badly and a diversion was necessary at the lower end, timber shoring being used to keep the water-course open at the upper end. Finally, it was decided to complete the tipping with ashes and similar light material. On the 1st August, 1933, a start was made with ashes which were delivered daily by two trains of thirty wagons each; 84,566 cubic yards of ashes were tipped and the embankment was successfully completed with ash filling on the 7th March, 1934, and on the next day the railway company started to prepare the permanent way for the main-line diversion, which took place without any hitch on Sunday, the 6th May. This ash top to the embankment is 20 feet deep over the maximum settlement in the clay embankment and has stood very well with hardly any settlement. The side slope is $1\frac{1}{2}$ to 1, with an extra 4 feet in width at the top on the cess over the contract section.

The slopes of the clay embankment which were then stabilized were dressed down by hand labour, soiled and sown. Rubble drains 3 feet by 3 feet and 2 feet by 2 feet were put in to intersect the surface, with 6-inch salt-glazed-ware pipes in some cases. These were thrust well into the clay under the ash toe, to drain as far as possible from a possible backfall in the surface of the tipped clay under the ash embankment. The side ditch at the toe of the embankment was relaid with 3-foot diameter reinforced-concrete pipes, covered with rubble to the ground surface, so that the toe of the embankment had solid ground against which to thrust, and the dingle at 10 miles 44 chains was also straightened out and the side wall strengthened. The opposing slope of the field between 10 miles 33 chains and 10 miles 41 chains was previously liable to slip, and the finish of the tipping to the new embankment settled up against these contra-slips and made the ground stable.

It appears that it would have been advisable to have tipped the embankment at a flatter slope, especially as this part of the countryside, which is in the Peak District, is liable to heavy rainstorms; it might also have been advisable to have bought more land between 10 miles 33 chains and 10 miles 41 chains, in order to tip out against the contra-slip area and so to stabilize the toe for the new embankment, although it would still have been necessary to provide a suitable retaining wall between 10 miles 32 chains and 10 miles 33 chains. A section of the main-line embankment as originally proposed and as finally completed is shown in Fig. 2, Plate 1.

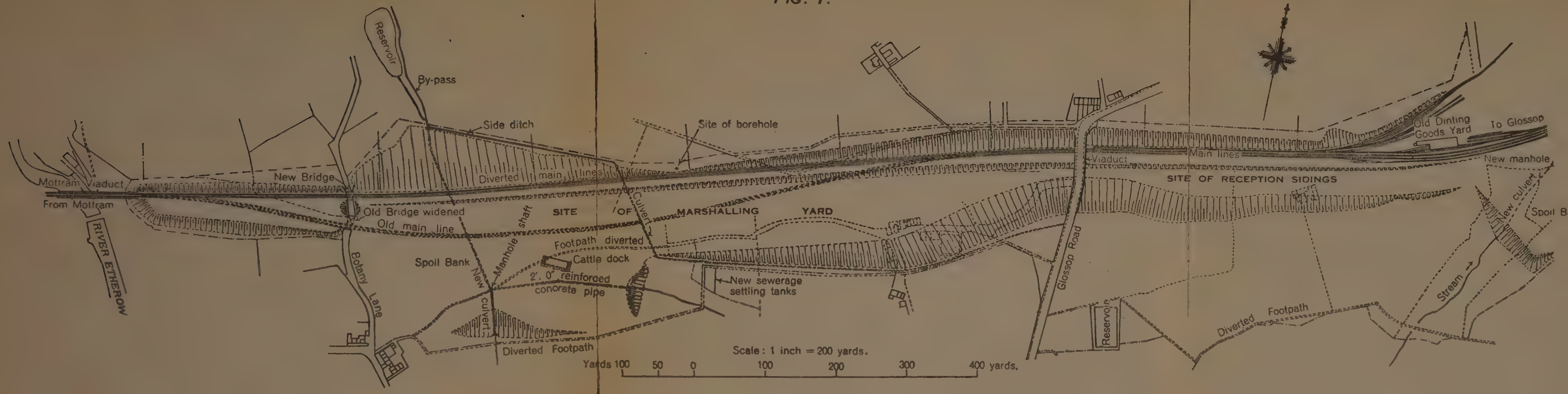
The total quantity of excavation was 1,201,350 cubic yards, of which 72,900 cubic yards carried from the up-side cutting, and 328,600 cubic yards from the down-side cutting, formed the main-line diversion embankment, whilst 799,850 cubic yards were carried to the spoil banks.

The final cost of the general earthwork, bridges and culverts was £171,956. The works were carried out under the direction of Mr. C. J. Brown, C.B.E., M. Inst. C.E., the Engineer for the Southern Area, and Mr. R. F. Bennett, M. Inst. C.E., Assistant Engineer (Construction) (who retired in December 1931), and later Mr. R. J. M. Inglis, M. Inst. C.E. The Resident Engineer was Mr. W. H. Watson until January, 1932, when the Author was appointed. The Contractors for the general earthwork, bridges and culverts were Messrs. H. Arnold & Son, Ltd., of Doncaster. Their Manager was Mr. Henry Peake, assisted by Messrs. Walter Peake and Edward Beckett, Assoc. M. Inst. C.E.

The Paper is accompanied by fourteen sheets of drawings, from some of which Plate 1 has been prepared.

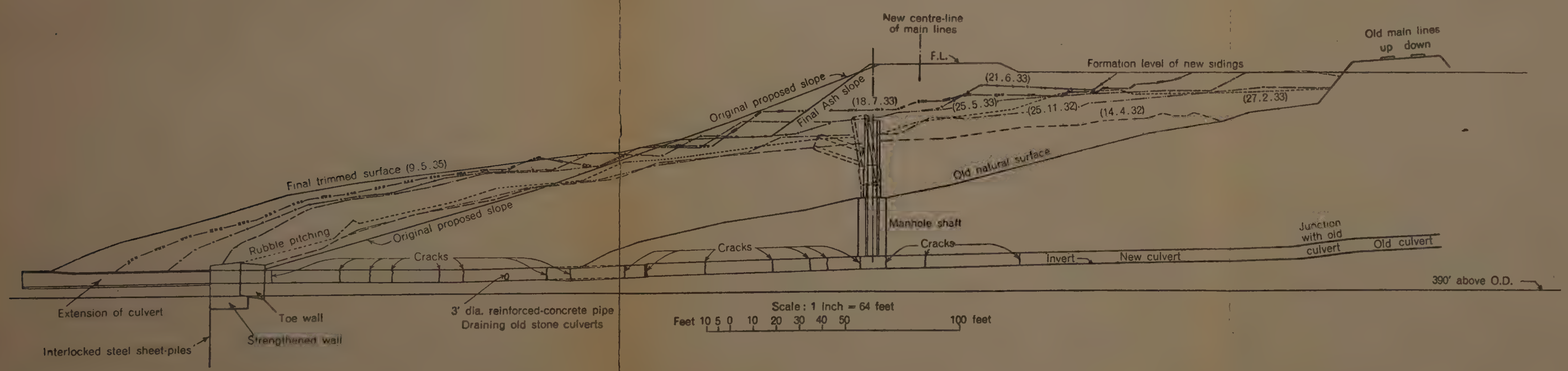
THE DIVERSION OF THE MAIN LINE BETWEEN MOTTRAM AND DINTING, L.N.E.R.

FIG: 1.



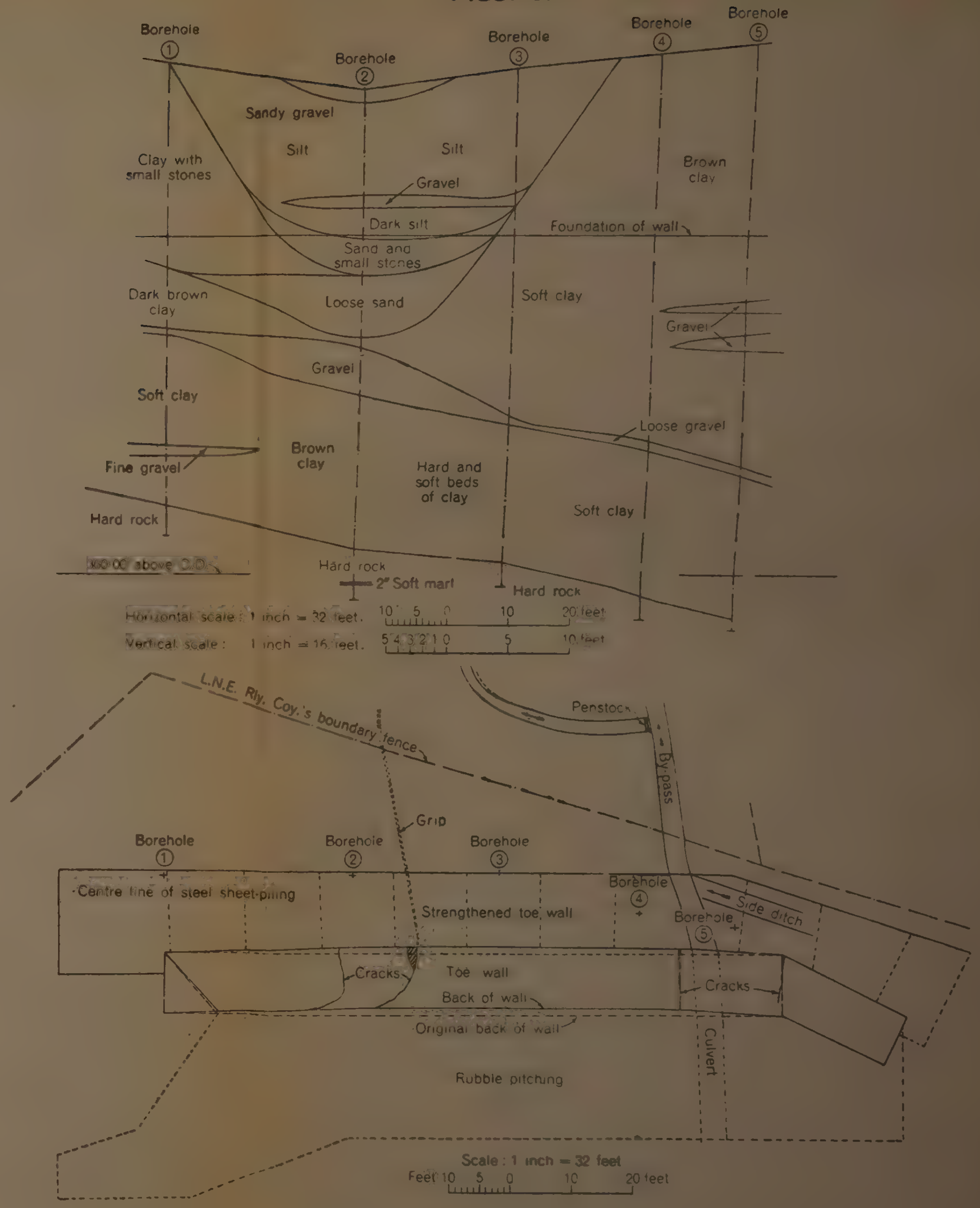
PLAN OF NEW YARD AND DIVERSION OF MAIN LINE.

FIG: 2.



SECTIONS THROUGH NEW MAIN-LINE EMBANKMENT, AND SECTION THROUGH NEW CULVERT.

FIGS: 3.



SECTIONS OF STRATA AND PLAN OF TOE WALL.

F. W. SLADE

WATERLOW & SONS LIMITED, LATE THOS. KELL & SON, LONDON.

Paper No. 5057.

"The Impedance of Transformers Connected in Cascade."

By Brian Laidlaw Goodlet, B.A., Assoc. M. Inst. C.E.

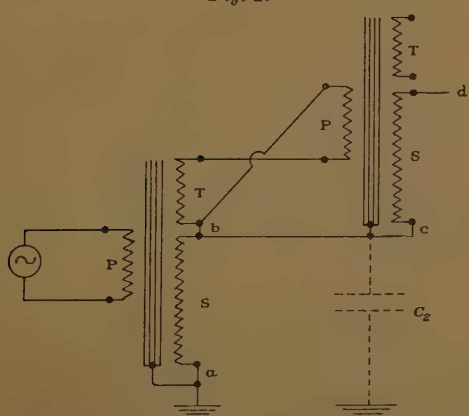
(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

Introduction.

The so-called "cascade" connexion of transformers is frequently employed for producing very high voltages for testing purposes. Two stages of such a cascade are shown in *Fig. 1*. Each transformer

Fig. 1.



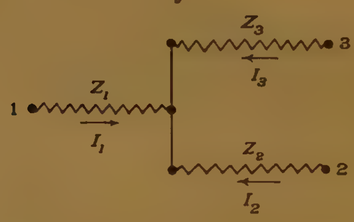
has three windings, termed primary, secondary and tertiary, and denoted in the Figure by P, S, and T. The primary and tertiary usually have about the same number of turns, and a working voltage

[1] Correspondence on this Paper can be accepted until the 15th May, 1937, and will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

of one or two thousand volts; the secondary windings have a much larger number of turns and generate several hundred thousand volts. The primary of No. 1 transformer is fed from a generator, whilst one end (a) of the secondary is permanently grounded. If E denotes the amplitude of the alternating voltage induced in the secondary, the potential of its terminal b will fluctuate between limits $\pm E$. The primary of No. 2 transformer is excited from the tertiary of No. 1, so that its secondary generates a similar voltage.

The two secondary voltages are added by connecting the secondaries in series as shown; the potential of point d therefore fluctuates between limits $\pm 2E$ to ground. To prevent the voltage from (d) to the frame of No. 2 unit from exceeding its own generated voltage E, this frame (core and tank) is insulated from ground and connected to the junction-point c. A similar "potential-anchoring" connexion is made from point b to the tertiary winding, which must

Fig. 2.



therefore be insulated for the full secondary voltage, although the voltage actually generated in it is much smaller. This method of connexion can be extended to three, four or more units. The problem examined in the present Paper is the calculation of the impedance and voltage-regulation of such a cascade, given the impedances of the individual units.

A Cascade of Two Units.

Consideration will first be given to the very common case of a cascade consisting of only two units, each generating say 500,000 volts. According to the usual theory, a three-winding transformer can be represented by a star of three impedances, as shown in *Fig. 2.* The assumptions on which this equivalent circuit is based are discussed in the Appendix. The definitive equations are:—

$$I_1 + I_2 + I_3 = 0,$$
$$E_1 - Z_1 I_1 = E_2 - Z_2 I_2 = E_3 - Z_3 I_3;$$

where E_1, E_2, E_3 denote the voltages at the terminals 1, 2, 3, and

I_1, I_2, I_3 denote the currents in the impedances Z_1, Z_2, Z_3 respectively. Using this representation the equivalent circuit of a 2-unit cascade is as shown in *Fig. 3*. The second unit of a 2-unit cascade does not require a tertiary winding, and this is accordingly omitted. The analysis proceeds as follows.

The general equations of the circuit are :

$$I_1 + I_2 + I_3 = 0, \quad \quad (1)$$

$$E_1 - Z_1I_1 = E_2 - Z_2I_2 = E_3 - Z_3I_3, \quad \quad (2)$$

$$I_4 + I_5 = 0, \quad \quad (3)$$

$$E_4 - Z_4I_4 = E_5 - Z_5I_5. \quad \quad (4)$$

The special conditions to be satisfied, owing to the connexions, are :

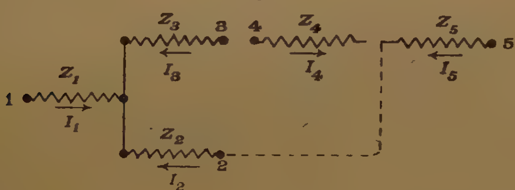
$$I_3 + I_4 = 0, \quad \therefore I_3 = -I_4; \quad \quad (5)$$

$$E_3 = E_4 . \quad \quad (6)$$

Hence, by (3) and (5), $\quad I_3 = I_5, \quad \quad (7)$

and, by (1), $\quad I_1 = -(I_2 + I_5). \quad \quad (8)$

Fig. 3.



To deduce an expression for the total voltage $(E_2 + E_5)$ of the cascade in terms of the applied generator-voltage E_1 and the output-current I_5, it is known that by (4)

$$E_5 = E_4 - Z_4I_4 + Z_5I_5 ;$$

by (5) and (6) this becomes

$$E_5 = E_3 + (Z_4 + Z_5)I_5.$$

But
$$E_3 = E_1 - Z_1I_1 + Z_3I_3 = E_1 + Z_1(I_2 + I_5) + Z_3I_5$$
$$= E_1 + Z_1I_2 + (Z_1 + Z_3)I_5. \quad \quad (9)$$

Hence
$$E_5 = E_1 + Z_1I_2 + (Z_1 + Z_3 + Z_4 + Z_5)I_5, \quad \quad (10)$$

and
$$E_2 = E_1 - Z_1I_1 + Z_2I_2$$
$$= E_1 + (Z_1 + Z_2)I_2 + Z_1I_5. \quad \quad (11)$$

Now the current I_2 in the secondary of No. 1 transformer will be the sum of the current I_5 in the secondary of No. 2 unit and the charging current drawn by the capacity of the connexion b—c (in *Fig. 1*) to ground. This capacity is formed by the capacity of No. 2 tank against ground and the winding and bushing capacity

of No. 1 secondary, and can be represented by a condenser of magnitude C_2, connected between points a and b. If

$$I_2 = I_5 + j\omega C_2 \,.\, E_2 (\text{where } j = \sqrt{-1}), \quad . \quad . \quad . \quad . \quad (12)$$

then
$$E_5 = E_1 + (2Z_1 + Z_3 + Z_4 + Z_5)I_5 + j\omega C_2 Z_1 E_2, \quad . \quad (13)$$

$$E_2 = E_1 + (2Z_1 + Z_2)I_5 + j\omega C_2 (Z_1 + Z_2)E_2, \quad . \quad . \quad (14)$$

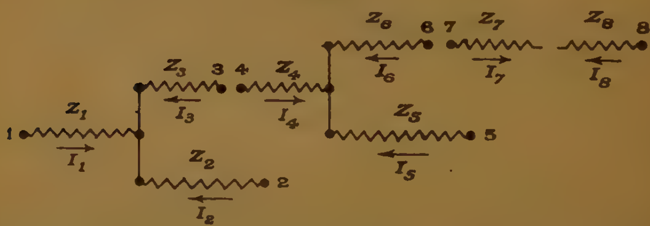
$$(E_2 + E_5) = 2E_1 + (4Z_1 + Z_2 + Z_3 + Z_4 + Z_5)I_5 + j\omega C_2 (2Z_1 + Z_2)E_2, \quad . \quad (15)$$

$$E_2 = \frac{E_1 + (2Z_1 + Z_2)I_5}{1 - j\omega C_2 (Z_1 + Z_2)}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

$$(E_2 + E_5) = E_1 \left\{ 2 + \frac{j\omega C_2 (2Z_1 + Z_2)}{1 - j\omega C_2 (Z_1 + Z_2)} \right\} + I_5 \left\{ 4Z_K + \frac{j\omega C_2 (2Z_1 + Z_2)^2}{1 - j\omega C_2 (Z_1 + Z_2)} \right\}, \quad . \quad (17)$$

where
$$Z_K = Z_1 + \frac{Z_2 + Z_3}{4} + \frac{Z_4 + Z_5}{4} \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

Fig. 4.



Equation (17) gives the required expression for the output-voltage $(E_2 + E_5)$ in terms of the generator-voltage E_1 and load-current I_5. The short-circuit impedance of the cascade can be found by putting $(E_2 + E_5) = 0$. If the charging current is neglected, then $I_2 = I_5$ and $I_1 = -2I_5$. Equation (15) then gives

$$I_1 = -2I_5 = \frac{4E_1}{4Z_1 + Z_2 + Z_3 + Z_4 + Z_5} = \frac{E_1}{Z_K}.$$

The short-circuit impedance is accordingly Z_K, defined by equation (18) above.

A Cascade of Three Units.

The procedure in this case (*Fig. 4*) is precisely similar. The general equations are:

$$I_1 + I_2 + I_3 = 0, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

$$E_1 - Z_1 I_1 = E_2 - Z_2 I_2 = E_3 - Z_3 I_3, \quad . \quad . \quad . \quad . \quad . \quad (20)$$

$$I_4 + I_5 + I_6 = 0, \qquad (21)$$
$$E_4 - Z_4 I_4 = E_5 - Z_5 I_5 = E_6 - Z_6 I_6, \qquad (22)$$
$$I_7 + I_8 = 0, \qquad (23)$$
$$E_7 - Z_7 I_7 = E_8 - Z_8 I_8. \qquad (24)$$

The special conditions arising from the connexions are:

$$I_6 + I_7 = 0, \qquad (25)$$
$$E_6 = E_7, \qquad (26)$$
$$I_3 + I_4 = 0, \qquad (27)$$
$$E_3 = E_4. \qquad (28)$$

By these relations all currents are expressed in terms of I_2, I_5, and I_8. Thus:

$$I_7 = -I_8, \qquad (29)$$
$$I_6 = I_8, \qquad (30)$$
$$I_4 = -(I_5 + I_6) = -(I_5 + I_8), \qquad (31)$$
$$I_3 = -I_4 = (I_5 + I_8), \qquad (32)$$
$$I_1 = -(I_2 + I_3) = -(I_2 + I_5 + I_8). \qquad (33)$$

By (24) and (29),

$$E_8 = E_7 + (Z_7 + Z_8)I_8,$$

and by (26) and (22)

$$E_7 = E_6 = E_4 - Z_4 I_4 + Z_6 I_6,$$

but $$E_4 = E_3 = E_1 - Z_1 I_1 + Z_3 I_3.$$

Substituting for I_1, I_3, I_4, and I_6,

$$E_8 = E_1 + Z_1 I_2 + (Z_1 + Z_3 + Z_4)I_5 + (Z_1 + Z_3 + Z_4 + Z_6 + Z_7 + Z_8)I_8. \qquad (34)$$

Similarly $E_5 = E_4 - Z_4 I_4 + Z_5 I_5 = E_1 + Z_1 I_2 + (Z_1 + Z_3 + Z_4 + Z_5)I_5 + (Z_1 + Z_3 + Z_4)I_8$, . (35)

and $E_2 = E_1 - Z_1 I_1 + Z_2 I_2$
$$= E_1 + (Z_1 + Z_2)I_2 + Z_1 I_5 + Z_1 I_8. \qquad (36)$$

Adding (9), (10), and (11) gives for the output voltage

$$(E_2 + E_5 + E_8) = 3E_1 + (3Z_1 + Z_2)I_2 + [3Z_1 + 2(Z_3 + Z_4) + Z_5]I_5 + [3Z_1 + 2(Z_3 + Z_4) + Z_6 + Z_7 + Z_8]I_8. \qquad (37)$$

Capacity-currents can be allowed for as before by writing

$$I_5 = I_8 + j\omega C_3(E_2 + E_5)$$

and $$I_2 = I_5 + j\omega C_2(E_2),$$

but the resulting expressions are too complicated to be given here and are best evaluated numerically. The short-circuit impedance can be found by equating (37) to zero with $I_2 = I_5 = I_8$. This gives

$$I_2 = \frac{-3E_1}{9Z_1 + 4(Z_3 + Z_4) + Z_2 + Z_5 + Z_6 + Z_7 + Z_8},$$

and, since by (33) $I_1 = -3I_2$, the short-circuit impedance will be

$$Z_K = Z_1 + \tfrac{1}{9}\{Z_2 + 4(Z_3 + Z_4) + Z_5 + Z_6 + Z_7 + Z_8\}. \quad . \quad (38)$$

The Paper is accompanied by four diagrams, from which the Figures in the text have been prepared, and by the following Appendix.

APPENDIX.

The Theory of the Three-Winding Transformer.

Consider an iron core carrying three concentric cylindrical windings of equal axial length. Let R denote the resistance, ϕ the total flux-linkage, and e the instantaneous voltage of any winding; then

$$\left.\begin{aligned} e_1 &= R_1 i_1 + \frac{d\phi_1}{dt}, \\ e_2 &= R_2 i_2 + \frac{d\phi_2}{dt}, \\ e_3 &= R_2 i_2 + \frac{d\phi_3}{dt}. \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Let ϕ_C denote the flux in the iron core, which, from the construction postulated, must link all three windings equally; then

$$\phi_1 = n_1\phi_c + \lambda_{11}i_1 + \lambda_{12}i_2 + \lambda_{13}i_3, \quad . \quad . \quad . \quad . \quad (2)$$

where n_1 denotes the number of turns in winding (1) and λ_{11}, λ_{12}, λ_{13} are coefficients of induction due to air fluxes. Hence

$$e_1 = R_1 i_1 + \lambda_{11}\frac{di_1}{dt} + \lambda_{12}\frac{di_2}{dt} + \lambda_{13}\frac{di_3}{dt} + n_1\frac{d\phi_C}{dt}, \quad . \quad . \quad . \quad . \quad (3)$$

with similar expressions for e_2 and e_3.

Let V denote the induced volts per turn due to ϕ_C: then, for a harmonic electromotive force of amplitude E,

$$\left.\begin{aligned} E_1 &= R_1 I_1 + jX_{11}I_1 + jX_{12}I_2 + jX_{13}I_3 + n_1 V, \\ E_2 &= R_2 I_2 + jX_{21}I_1 + jX_{22}I_2 + jX_{23}I_3 + n_2 V, \\ E_3 &= R_3 I_3 + jX_{31}I_1 + jX_{32}I_2 + jX_{33}I_3 + n_3 V, \end{aligned}\right\} \quad . \quad . \quad . \quad (4)$$

where X_{11}, X_{12}, etc., denote mutual and self-reactances due to air fluxes. Now, writing

$$\left.\begin{aligned} R_2' &= \left(\frac{n_1}{n_2}\right)^2 R_2, & R_3' &= \left(\frac{n_1}{n_3}\right)^2 R_3, \\ I_2' &\times \left(\frac{n_2}{n_1}\right) I_2, & I_3' &= \left(\frac{n_3}{n_1}\right) I_3, \\ E_2' &= \left(\frac{n_1}{n_2}\right) E_2, & E_3' &= \left(\frac{n_1}{n_3}\right) E_3, \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (5)$$

then

$$E_2' = R_2'I_2' + jX_{21}\frac{n_1}{n_2}I_1 + jX_{22}\left(\frac{n_1}{n_2}\right)^2 I_2' + jX_{23}\frac{n_1^2}{n_2 n_3}I_3' + n_1 V, \quad . \quad (7)$$

$$E_1 = R_1 I_1 + jX_{11}I_1 + jX_{12}\frac{n_1}{n_2}I_2' + jX_{13}\frac{n_1}{n_3}I_3' + n_1 V, \quad . \quad . \quad . \quad . \quad (6)$$

$$E_3' = R_3'I_3' + jX_{31}\frac{n_1}{n_3}I_1 + jX_{32}\frac{n_1^2}{n_2 n_3}I_2' + jX_{33}\left(\frac{n_1}{n_3}\right)^2 I_3' + n_1 V. \quad . \quad (8)$$

Now the algebraic sum of the currents must equal the magnetizing current I_m, so that

$$I_1 + I_2' + I_3' = I_m,$$

or

$$I_3' = I_m - I_1 - I_2' \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

Substituting this value of I_3' in (6) and (7),

$$E_1 = R_1 I_1 + j\left[X_{11} - X_{13}\frac{n_1}{n_3}\right]I_1 + j\left[X_{12}\frac{n_1}{n_2} - X_{13}\frac{n_1}{n_3}\right]I_2' + jX_{13}\frac{n_1}{n_3}I_m + n_1 V,$$

$$E_2' = R_2'I_2' + j\left[X_{21}\frac{n_1}{n_2} - X_{23}\frac{n_1^2}{n_2 n_3}\right]I_1 + \left[X_{22}\left(\frac{n_1}{n_2}\right)^2 - X_{23}\frac{n_1^2}{n_2 n_3}\right]$$

$$I_2' + jX_{23}\frac{n_1^2}{n_2 n_3}I_m + n_1 V.$$

Subtracting,

$$E_1 - E_2' = \left\{R_1 + j\left[X_{11} - X_{13}\frac{n_1}{n_3} - X_{21}\frac{n_1}{n_2} + X_{23}\frac{n_1^2}{n_2 n_3}\right]\right\}I_1$$

$$+ \left\{R_2' + j\left[X_{22}\left(\frac{n_1}{n_2}\right)^2 + X_{13}\frac{n_1}{n_3} - X_{12}\frac{n_1}{n_2} - X_{23}\frac{n_1^2}{n_2 n_3}\right]\right\}I_2'$$

$$+ j\left\{X_{13}\frac{n_1}{n_3} - X_{23}\frac{n_1^2}{n_2 n_3}\right\}I_m \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Precisely similar equations can be found for $E_2 - E_3$ and $E_3 - E_1$. Writing

$$Z_1 = R_1 + j\left[X_{11} - X_{13}\frac{n_1}{n_3} - X_{21}\frac{n_1}{n_2} + X_{23}\frac{n_1^2}{n_2 n_3}\right], \quad . \quad . \quad . \quad (11)$$

$$Z_2 = R_2' + j\left[X_{22}\left(\frac{n_1}{n_2}\right)^2 + X_{13}\frac{n_1}{n_3} - X_{12}\frac{n_1}{n_2} - X_{23}\frac{n_1^2}{n_2 n_3}\right], \quad . \quad (12)$$

$$Z_3 = R_3' + j\left[X_{33}\left(\frac{n_1}{n_3}\right)^2 - X_{13}\frac{n_1}{n_3} + X_{12}\frac{n_1}{n_2} - X_{23}\frac{n_1^2}{n_2 n_3}\right], \quad . \quad (13)$$

and assuming that the magnetizing current $I_m = 0$, then

$$\left.\begin{aligned} E_1 - E_2' &= Z_1 I_1 - Z_2 I_2' \\ E_2' - E_3' &= Z_2 I_2' - Z_3 I_3' \\ E_3' - E_1 &= Z_3 I_3' - Z_1 I_1 \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

or

$$E_1 - Z_1 I_1 = E_2' - Z_2 I_2' = E_3' - Z_3 I_3' \quad . \quad . \quad . \quad . \quad (15)$$

This is the voltage-equation for the impedance star of *Fig. 2*, with the currents in the sense shown. This star is accordingly the equivalent circuit of a three winding transformer on the assumptions stated.

To determine the impedances Z_1, Z_2, Z_3 by test, the short-circuit impedances

between each pair of windings must be measured. Let these three values be denoted by Z_{12}, Z_{23}, Z_{31}; then

$$Z_{12} = Z_1 + Z_2$$
$$Z_{23} = Z_2 + Z_3$$
$$Z_{31} = Z_3 + Z_1$$

whence

$$Z_1 = \frac{Z_{12} - Z_{23} + Z_{31}}{2},$$

$$Z_2 = \frac{Z_{23} - Z_{31} + Z_{12}}{2},$$

$$Z_3 = \frac{Z_{31} - Z_{12} + Z_{23}}{2}.$$

One of these impedances is usually negative, for reasons which cannot be discussed here.

It is important to recognize that the equivalent circuit of *Fig. 2* is valid only on the assumptions (*a*) that all three windings are linked by a common core-flux which induces the same volts per turn in each winding, and (*b*) that the sum of the ampere-turns on the core is zero.

The first assumption is necessarily fulfilled when the three windings are three concentric cylindrical coil-stacks of equal height, jacketing the same length of core; if the coil-stacks are of different length or are on different parts of the core the core-flux is quite likely to vary. The second assumption is never exactly true, and may be seriously in error if the magnetizing current is appreciable owing to the core being saturated or because air-gaps exist in the magnetic circuit.

Paper No. 5068.

"Moving a Tower Crane in Connexion with the Building of the Q.S.S. 'Queen Mary.'"

By Robert Dunlop Brown, Jun., B.Sc., Stud. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*)[1]

Just before the launch of the Cunard—White Star quadruple-screw liner "Queen Mary" the crane which was in use alongside the launching-ways had to be moved through a distance of 101 feet 2 inches to a new position alongside the fitting-out basin, and, in order to save the time which would otherwise be necessary for dismantling, it was decided to move the crane bodily. The crane was of the cantilever or "hammer-head" type and capable of lifting 10 tons at a radius of 160 feet. It weighed 200 tons and was about 200 feet high.

In this Paper the methods used in the moving of the crane are described in detail. Briefly the procedure was as follows :—

After carefully balancing the structure, the crane was lifted off its old foundations through a height of 32½ inches by means of four 200-ton hydraulic jacks. Timber launching-ways were placed in position under the four legs of the crane, leading to the new foundation-blocks, and the crane was lowered on to timber sliding-ways standing on the launching-ways. By means of an electric winch, the crane was then hauled very slowly along the ways to its new position, where it was lifted up slightly and the ways removed. The crane was finally lowered on to its new foundations.

The moving of the crane was greatly facilitated by the employment of men who were accustomed to the methods employed in launching ships. The actual hauling of the crane along the ways, which had been coated with wax and soft soap, was, however, so easy that the hauling-tackle did not even lift up off the ways. The operation of raising, moving and lowering the crane on to the new position, after the completion of the new foundations, took 3 days.

The Author is indebted to Messrs. Sir William Arrol & Co., Ltd., who were the contractors, for their kind permission to submit the Paper.

---

[1] The MSS. and drawings can be seen in the Institution Library.

Paper No. 4977.

"The Treatment of Mud-Runs in Bolivia."

By STEPHEN WILLIAM FRANCIS MORUM, B.Sc. (Eng.), Assoc. M. Inst. C.E.

Author's Reply to Discussion and Correspondence.[1]

The AUTHOR wished to express his thanks for the interest that had been shown in his Paper, and regretted that he had not been able to present a record of more than a few preliminary experiments.

Replying to Mr. Carpmael, he had himself noticed that some of the inspectors he had had to deal with showed a certain reluctance to treat slips at their source, the general tendency being to clear the track without taking preventive measures. "Mud-runs," however, should not be confused with slips, for, whilst slips in a valley-side were the primary cause of the runs, the detritus would never reach the track unless it were brought down by the mountain streams in spate.

The annual rainfall over the part of Bolivia where mud-runs principally occurred varied considerably, but might be said to lie between 25 and 35 inches per annum. Whilst the rainy season was from November to April, the majority of the rain fell between mid-December and mid-March, the falls during that period being of the order of 10 inches per month, though falls greatly in excess of that figure had been recorded. The rains coincided with the hot season, and were more of the nature of tropical downpours than of rainy days as experienced in the British Isles; it was usual to have four or five wet days, and then probably a spell of fine weather broken only by showers, local tradition holding that the spells were dependent on the moon. The rainfall on wet days at the height of the season usually consisted of a steady rain with heavier showers and brighter intervals, but it was impossible to generalize. Falls of 4 inches during 24 hours were quite usual, and 6 inches had sometimes been recorded. The actual mud-runs, which depended on a large volume of water, occurred during and after the heavy downpours; but the slides which were their primary cause were more prevalent

[1] The Paper, Discussion, and Correspondence have already been published: Journal Inst. C.E., vol. 1 (1935–36), pp. 426 and 564*. (January and October numbers respectively.)

after a day or so of steady rain, when the ground had become thoroughly soaked and lubricated. Slips in general, as mentioned by Mr. Carpmael, were a further serious maintenance-problem in the Andes, but were beyond the scope of the Paper.

He wished to thank Dr. Lowe-Brown for his kind remarks, but would say that much of the credit for the preventive work he had described was due to Mr. F. F. Williams, who had initiated the major part of it. He thought Dr. Lowe-Brown had misunderstood his description of the mountainous conditions of Bolivia, as when giving the slopes he had intended to describe the slope of the ravine-bed and not of the actual hill-sides, which were more of the nature of those in Borneo described by Dr. Lowe-Brown; that could be seen from a study of *Figs. 9, 10,* and *11* of the Paper.

He had read with much interest Sir Gordon Hearn's account of the construction of the railway through the Khyber pass; he was struck by the similarity of many of the problems encountered, and agreed that the difficulties confronting a location-engineer in such countries had to be seen to be believed. The more detailed description given of some of the works was extremely interesting. In regard to Sir Gordon's criticism of the Orcoma deviation-channel (*Fig. 3*), any other location would have entailed a more costly job, and would also have cut down the deposit-space and the mass available for the retention of the ravine-sides. The cost of importing iron tubes was such as seriously to restrict their use; owing to the erosive power of the detritus-charged streams he did not think that corrugated iron tubes would have had a very long life. In that particular case it had not been possible to carry the deviation over the spur, as the ground there was not good enough, but he considered that that method should be further developed if the cost were not excessive; however, as Sir Gordon had pointed out, the water-courses varied to such an extent that generalization was impossible. He agreed that his estimation of the rainfall-intensity was over rather a long period, but owing to the difficulties of obtaining records a shorter period had not been practicable. Regarding the E. A. Moritz formula, he understood that it had been primarily developed to define the critical mean velocity, for a given depth, above which a stream picked up silt. For an ideal case with no erosion in the channel the velocity would need to be below that figure, but if detritus had to be carried with a minimum flow, the velocity of flow had to be made high enough to avoid silting. A compromise then had to be made by some means such as paving the channel. He regretted that a slip had been made in the Paper (p. 457): the formula had actually been used in the case of unpaved channels of a softish nature.

He agreed with Mr. Gribble that the methods described had not been in operation long enough to show whether they were permanent or not, and that the best solution would be to locate new railways so as to avoid the difficulties described, but he would point out that the construction-costs would probably be increased out of all proportion, and that in the cases described the problem to be faced had been that of maintaining an existing line.

He was interested to note the similarity of the works described by Mr. Williamson, which had been in operation for some while, and thanked him for the description.

In reply to Mr. Blencowe, whilst agreeing that in general it would be better to locate the line as suggested, he felt that it was often more necessary, particularly in the Andes, to pay close attention to the side ravines, as owing to the contorted and faulted nature of the countryside a dangerous side ravine was often found on side B (*Fig. 13*).[1] Much of the trouble of maintenance would undoubtedly be saved by skilful location.

He was very interested by Mr. Lavis's account of the Atocha—Tupiza line, as he had several times travelled over it; he believed that in regard to absence of *mazamorras* and similar troubles it was one of the most successful lines in Bolivia, and it was certainly most interesting. He agreed that it was far easier to criticize a location, especially in such difficult country as the Andes, than to make one. He felt, however, that there had been a tendency to disregard the *mazamorras* in the earlier locations, and also that Mr. Lavis would agree that if the Cochabamba line had been located higher after leaving Colcha much of the *mazamorra* trouble would have been avoided. The line would certainly, however, have been more costly in the first instance, and possibly other difficulties would have had to have been overcome; he agreed that it was impossible in the case of the Bolivian branch-lines to leave the river-valleys. He also agreed that a practical treatment was often of more value than a theoretical one, but felt that if such practical work had a theoretical backing, it had an increased value especially to an engineer confronted with a problem that was new to him.

In reply to Mr. Legget, he regretted that he had no geological section available, and, as he had left the country, no means of making one, but he agreed that a geological study would be of assistance in approaching such problems. He would point out, however, that in country as varied as Bolivia there could never be a typical section, and that the maintenance-engineer seldom had the time available to devote to a geological survey, which unless made in

[1] Vol. 1, p. 565*. (October, 1936.)

detail and with care would tend to defeat its own ends. Mr. Legget was correct in assuming that one of the objects of the dams was to form a series of flatter slopes; that was particularly so in the case of Type 1 walls. The other types of dams were used partly with that object and partly to prevent the sides of the ravines sliding, by providing them with a firm toe, and thus stopping erosion. The check-dams were only heightened during their initial season, or when settlement made it necessary to do so; it was customary rather, if necessary, to build additional ones than to risk the possibility of failure through too great a height.

He regretted that he had no figures available with regard to the abrasive power of the runs, but it would be realized that a mixture of water, grit and stones moving at a comparatively high velocity could do considerable damage in a short time.

NOTE.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Sub-Committee on Soil Corrosion of Cement Products.

In the December Journal the formation of a Committee on Soil Corrosion of Metals and Cement Products was announced. This Committee supersedes the former Sub-Committee on the Effect of Soils containing Sulphate Salts on Concrete and Metal Pipes. A considerable amount of preliminary work had, however, already been done in connexion with the corrosion of concrete; a survey had been made of existing information, the occurrence of deterioration over the whole of Great Britain had been studied, and a tentative programme of experimental research had been drawn up. This programme envisaged an expenditure of about £10,000 on a long-period research extending over about 10 years. The appeals for financial and material support in the carrying out of this research had resulted in the promise of a considerable amount of assistance.

In these circumstances it was decided to form concurrently with the main Committee on Soil Corrosion a Sub-Committee dealing with the Soil Corrosion of Cement Products. The composition of the new Sub-Committee, which includes among its members consulting engineers, a county surveyor, chemists, a contractor, cement manufacturers and pipe manufacturers, is as follows:—

R. G. Hetherington, C.B., O.B.E., M.A. (*Chairman*).
E. M. Crowther, D.Sc., F.I.C.
E. A. Dancaster, Ph.D., M.Sc., F.I.C.
C. L. Howard Humphreys, T.D.
J. G. Kay.
F. M. Lea, D.Sc.
Alexander Melville.
S. G. S. Panisset.
W. P. Robinson.
R. H. H. Stanger.
Godfrey Taylor, M.C., M.A.
N. B. Walker.
J. D. Watson.
P. H. Wilson.

The financial support promised to the former Sub-Committee has been in all cases generously transferred to the new Sub-Committee, and the latter is now engaged in the drawing up of a final programme of research.

Sub-Committee on Repeated Stresses in Structural Elements.

Whilst a large amount of experimental evidence is available with regard to the reduction in the ultimate strength of materials under repeated stresses, little work has been done on the effect of such stresses on built-up elements including riveted and welded joints. Some preliminary tests[1] carried out by Professor F. C. Lea at Sheffield University in a specially-designed testing machine, whereby the structural element was subjected to repetitions of constant bending-strain, indicated that the ultimate strength of a jointed element, whether welded or riveted, was considerably less than that of the plain beam.

In view of the practical significance of these results the Department of Scientific and Industrial Research have made a grant towards a continuance of this research for a further 2 years and The Institution also has decided to give its assistance both financially and in a consultative capacity. A Sub-Committee of the Research Committee has accordingly been formed with the following personnel:

R. E. Stradling, C.B., M.C., Ph.D., D.Sc. (*Chairman*).
David Anderson, B.Sc., LL.D.
H. J. Gough, M.B.E., D.Sc., Ph.D.
Professor F. C. Lea, O.B.E., D.Sc. (Eng.).
Professor A. J. Sutton Pippard, M.B.E., D.Sc.
Professor Andrew Robertson, D.Sc.

The programme of research drawn up by Professor Lea, on which a start has already been made at Sheffield University, consists of a main series of repeated bending-tests together with subsidiary static bending-tests and tensile, impact, hardness and bending-tests on specimens cut from the girders. The following series of girders have been prepared for test:—

Welded joists 5 inches × 3 inches, normalized.
Mild steel girders 5 inches × 3 inches, plain, as rolled.
Mild steel girders 5 inches × 3 inches, rivet-holes in the tension flange.
Mild steel girders 5 inches × 3 inches, butt-welded joint in the centre.
Mild steel girders 5 inches × 3 inches, coverplate-welded joint in the centre.
Mild steel girders 5 inches × 3 inches, coverplate-riveted joint in the centre.

[1] F. C. Lea, "Repeated Stresses on Structural Elements," Journal Inst. C.E., vol. 4 (1936–37), p. 93. (November 1936.)

The experimental investigations at Sheffield cover only one aspect of the Sub-Committee's work, and it is hoped that the whole subject of fatigue in structural connexions will be considered in detail.

RESEARCH WORK IN ENGINEERING AT BATTERSEA POLYTECHNIC INSTITUTE, DECEMBER, 1936.

The following is a brief description of the researches in progress in the Departments of Civil and Mechanical Engineering, and of Electrical Engineering.

Civil and Mechanical Engineering Department.

An investigation into the production and properties of vibrated concrete is being made by an apparatus in which the vibration is produced by electro-magnetic means, with the result that the speed of vibration and amplitude can be regulated at will and the rate at which energy is absorbed can be studied during the process. The phenomena associated with the "healing" of concrete after cracking due to excessive loading are being studied. It is found that concrete cylinders loaded to incipient failure at 7 days not only recover their strength but after maturing in water may attain much higher strengths.

Several researches deal with reinforced concrete. An experimental investigation has been made of the strength of square and rectangular reinforced-concrete slabs supported on columns and free at the edges, carrying uniformly-distributed loads. The load is applied as a water-pressure, a rubber seal being fitted around the edge of the plate. Similar experiments have been carried out using steel plates as a check on the reinforced-concrete experiments and for comparison with theoretical calculations of deflexion. A further research on reinforced concrete is being made into the strength of right-angle connexions and the effect of various arrangements of reinforcement at the angle.

Another research concerns the strength and rigidity of rolled-steel joists loaded in torsion. The stresses induced at the roots of rolled-steel sections when subjected to a twisting moment do not readily lend themselves to mathematical computation. An investigation is being made into the failure of long struts of angle section due to elastic torsional instability. The strength and rigidity of girders having plate webs bent in such a way that a longitudinal section is zig-zag in form are being investigated and compared with the rigidity of corresponding girders with plain webs. Little is known about the elastic properties of wire ropes and a research is being carried out

into elastic and hysteretic phenomena associated with wire ropes; in particular their bearing on the design of ships' derrick cranes is being studied.

Researches dealing with hydraulics include an investigation of the effect of varying the angle on the flow over notches of trapezoidal section. By combining a notch of a particular trapezoidal form with an orifice of a certain size the flow can be made to vary linearly with the head, an arrangement particularly suitable for automatic recording. The best design and layout of a venturi flume is also being investigated. A study is being made of the loss of head and the resistance to flow offered by piers and obstructions of various shapes in a flume of rectangular section.

A certain amount of research is being carried out into problems connected with high-speed diesel engines. A method is being developed by which piezo-electric crystals may be used for the determination of pressure-variations in the cylinder of a high-speed internal-combustion engine. The piezo-electric crystals give a measure of the strain in the cylinder walls, and this is being correlated with the internal pressure.

Electrical Engineering Department.

In a recent research a method has been evolved by which the indicial admittance of electrical networks such as are used in television and sound-transmission systems can be measured. All such circuits have been classified into a few types, the characteristics of which can be expressed by mathematical formulas.

A research is in progress into the effects of magnetic polarization on the magnetic properties of the special alloy-steels used in measuring-instrument transformers. It is found that errors are introduced as a result of over-excitation with consequent residual magnetism. The extent of this effect has been measured by both the ballistic galvanometer and the alternating-current potentiometer. Magnetic hysteresis is being studied by means of the cathode-ray oscillograph. The magnetizing effect of alternating current is less than that of direct current of the same magnitude in consequence of the eddy-currents set up, but at high current-densities this difference appears to die out. An investigation is being made into the magnetic stability of modern alloy-steels, that is to say, their suitability as permanent magnets. Experiments are being carried out on steels containing chromium, cobalt and aluminium, and nickel and cobalt in combination.

A research which is being carried out on thyratrons is directed towards the production of a stable output under conditions of varying load and power-factor.

The above researches are being carried out under the direction of Mr. V. C. Davies, B.Sc. (Eng.), head of the Department of Civil and Mechanical Engineering, and Mr. A. T. Dover, head of the Electrical Engineering Department.

NOTES ON RESEARCH PUBLICATIONS.

Measurement.

The measurement of rapidly-alternating strains with a capacity extensometer, in which small rapid changes in length are transformed into changes of capacity and recorded by an oscillograph, is described in *Bauing.*, **17,** *98.*

Engineering Materials: Properties and Testing.

The Committee of The Institution on the Deterioration of Structures of Timber, Metal and Concrete exposed to the action of Sea-water have issued their 16th (Interim) Report.

Timber.

A handbook of home-grown timbers has been issued by the Forest Products Research Board of the Department of Scientific and Industrial Research. The causes and detection of brittleness in wood are dealt with in *Trade Circular No. 32, Division of Forest Products, Council for Scientific and Industrial Research, Australia.*

Cement and Concrete.

Recent researches on concrete for roads, bridges and dams are discussed in *Ver. deutsch. Ing.*, **80,** *1129.* A study of the quality, the design and the economy of concrete is given in *J. Franklin Inst.*, **221,** *495*, *653*, *745*, and **222,** *83.* *Building Research Bulletin No. 15* has been issued on the subject of lightweight concrete aggregates. A publication on special cement for use in large dams has been issued under Question III by the Second Congress on Large Dams of the World Power Conference. An account of an investigation into irregularities in the setting of cements is given in *Mitteilungen aus den Forschungsanstalten der Gutehoffnungshütte-Konzern*, **4,** *157.* In *J. Am. Concr. Inst.*, **8,** *41*, is an article on the properties of job-cured concrete at early ages. Shrinkage tests of grout mortars are described in *Technical Memo. No. 520, U.S. Bureau of Reclamation.* Temperature effects in mass concrete are dealt with in a publication

of the Second Congress on Large Dams of the World Power Conference. A summary of current literature on permeability and durability of concrete is given in *Water & Water Engineering*, **38,** *445*. The behaviour of concrete with and without protective coatings in marshy soils is dealt with in *Zement*, **25,** *601*, and a review of existing knowledge in regard to soils injurious to concrete is given in *Geologie und Bauwesen*, **8,** *12*.

Metals.

The recrystallization and ageing of mild steel is dealt with in *Het Schip*, **18,** *272*. The final report on an investigation of the practical problems of corrosion, describing some tests on protective coatings, is given in *Soc. Chem. Ind. J.*, **55,** *337*. An article on the atmospheric rusting of iron appears in *Mitteilungen der Deutsche Materialprüfungsanstalten*, **28,** *27*. Electrolytic corrosion in electric installations is dealt with in *Elek. Zeit.*, **57,** *1345*, and electrolytic measurement of the corrosiveness of soils in *U.S. Bur. Stand. J. Research*, **17,** *363*.

Other Materials.

The modulus of elasticity, strength and extensibility of refractories in tension are dealt with in *U.S. Bur. Stand. J. Research*, **17,** *463*, and a note on the modulus of elasticity of some fired brick-clay units appears in a preprint of *Ceram. Soc. Trans.*, *1936*, p. *4*.

ENGINEERING MATERIALS: PRODUCTION, MANUFACTURE AND PRESERVATION.

Metals.

The Research Association of British Paint, Colour and Varnish Manufacturers have issued *Bulletin No. 16*, dealing with the preservation of iron and steel by means of paint.

Other Materials.

A summary of information regarding the production and application of those organic plastic materials which are of chief industrial significance is given in *U.S. Bur. Stand. Circular No. C411*.

STRUCTURES.

Mass Structures.

The cumulative effect of inverted arch action in soil and other loose materials, as a result of the filling in and ramming of pipe trenches is discussed in *International Association for Bridge and Structural Engineering, Publications*, **4,** *359*. The effect of subsoil

discontinuities upon the pressure-distribution beneath a load is dealt with in *Science et Ind.*, **20,** *226*. Experimental work to determine the distribution of shear stresses in concrete floor slabs under concentrated loads is described in *Iowa Engineering Experiment Station, Bulletin No. 126*. A simplified rational pile-driving formula is described in *Science Reports of National Tsing Hua University, Series A*, **3,** *367*. The resistance of pile foundations to horizontal stresses is dealt with in *Stroitelnie Promishlennost* (*Moscow*), **14,** *37*. The following publications have been issued in connexion with the Second Congress on Large Dams of the World Power Conference: Question III: Special Cements. Question IV: Design and Waterproofing of Shrinkage, Contraction and Expansion Joints. Question V: Study of Facing of Masonry and Concrete Dams. Question VI: Geotechnical Studies of Foundation Materials. Question VII: Calculation of the Stability of Earth Dams. Communication No. 2: Dams Built of Precast Concrete Blocks.

The stresses around circular holes in dams and buttresses are analysed in *Am. Soc. Civ. Eng. Proc.* **62,** *1361*. The exact theory of thick cylindrical shells is developed in *International Association for Bridge and Structural Engineering, Publications* **4,** *131*.

Framed Structures.

Temperature-stresses in flat rectangular plates and in thin cylindrical tubes are discussed in *J. Franklin Inst.*, **222,** *149*. Tests on circular dished end flanges for cylindrical pressure-vessels are given in *Wärme*, **59,** *709*. A mathematical investigation of containers with plane walls is described in *Bauing.*, **17,** *40*, and in the same publication, p. *81*, the design of the web in the end bay of a plate girder is dealt with. Research on reinforced concrete includes an investigation of the stresses in shear reinforcement of reinforced-concrete beams, *Engineering*, **142,** *269*; an article on the modular ratio, *Beton und Eisen*, **35,** *324*; and a study of reinforcement in concrete slabs, *Am. Concr. Inst. J.*, **8,** *1*. Tests on concrete columns with rolled sections as reinforcement and on beam connexions to such columns are described in *Deutscher Ausschuss für Eisenbeton, Heft 81*. Research on the strength of windows is described in *Phil. Mag.*, **2,** *1165*.

Transformation, Transmission and Distribution of Energy.

The following electrical researches have been noted: The balancing of rotors by means of electrical networks, *J. Franklin. Inst.*, **222,** *183*; The protection of transformers by choking coils, *Elek. Zeit.*, **57,** *1262*; The variation of the magnetic properties of ferromagnetic

laminæ with frequency, *J. Inst. Elec. Engineers*, **79**, *667*; and The compensation of harmonics in the earth currents of three-phase systems, *Brown-Boveri Review*, **23**, *231*.

Mechanical Processes, Appliances and Apparatus.

In connexion with research on welding, a paper on the formation of metallic nitrides in the welding of steels is given in *Paper No. 20*, of the *XIIth International Congress of Acetylene, Oxy-Acetylene Welding and Allied Industries, 1936*. The effect of weld penetration on the stress in fillet welds is dealt with in *Am. Weld. Soc.*, **15** (*9*) *Supplement* p. *13*, and in the same journal, (*10*) *Supplement* p. *2*, the effect of low temperature on the tensile impact resistance of welded joints is discussed. The distribution of stresses in fillet welds is dealt with in *J. Inst. Eng. Australia*, **8**, *286*.

Specialized Engineering Practice.

Transport.

The Department of Scientific and Industrial Research has published *Road Research Bulletin No. 2*, The shape of road aggregate and its measurement, and *Technical Paper No. 4*, The control of the moisture content of aggregates for concrete, introducing a new vibration method. The relation between absolute viscosity and penetration of asphaltic bitumens is dealt with in *Physics*, **7**, *408*. The horizontal motion of marine structures under tidal action is discussed in *Zeit. Vermessungswesen*, **65**, *570*.

Research in connexion with air transport includes an article on the solution of the tail-less aeroplane problem by the Fauvel " flying wing," *Aéronautique*, **18**, *178*, and the following *Aeronautical Research Committee Reports and Memoranda; No. 1703*, Full-scale experiments at high angles of incidence with a IIIF seaplane; and *No. 1704*, Structure of turbulence in a natural wind, with a description of a sensitive pressure-gauge.

Research in connexion with hydro-electric engineering includes An experimental study of the scour of a sandy river bed by clear and by muddy water, *U.S. Bur. Stand. J. Research*, **17**, *193*, and a description of the desilting works of the Pont-de-Claix and the Drac-Inférieur water-power installations, *Bull. Tech. de la Suisse Romande*, **62**, *269*.

Of interest in connexion with mining research are some tensile tests on worn wire ropes, described in *U.S. Bur. Stand. J. Research*, **17**, *401*.

Research publications on acoustics include an article on the

vibration of a building partition at audio-frequencies, *Proc. Phys. Soc.*, **48**, *914*, and in the same journal, p. *919*, the vibration pattern of a wall transmitting sound.

Miscellaneous.

The following mathematical papers on fluid friction between rotating cylinders have been noted: *Proc. Roy. Soc.*, **157**, p. *526*, The forces on a circular cylinder submerged in a uniform stream; p. *546*, Torque measurements; and p. *565*, Distribution of velocity between concentric cylinders when the outer one is rotating and the inner one is at rest; and in the same journal, p. *594*, is described an experimental research into the movement of desert sand.

MADE AND PRINTED IN GREAT BRITAIN BY WILLIAM CLOWES AND SONS, LIMITED, LONDON AND BECCLES.